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MODERN
TIMBER
DESIGN

BY HOWARD J. HANSEN

TIMBER ENGINEERS' HANDBOOK
Cloth; 5½ by 8⅜; 882 pages;
256 illustrations.

MODERN TIMBER DESIGN
Second Edition. Cloth; 5⅞ by 9;
312 pages; 174 illustrations.

MODERN TIMBER DESIGN

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Professor of Mechanics, University of Florida

Second Edition

1948

NEW YORK · JOHN WILEY & SONS, INC.
LONDON · CHAPMAN & HALL, LIMITED

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SECOND EDITION
Second Printing, August, 1949

PRINTED IN THE UNITED STATES OF AMERICA

PREFACE TO THE SECOND EDITION

Shortly after the publication of the first edition, the Conservation Division of the War Production Board issued Directive 29, known as "National Emergency Specifications for the Design, Fabrication, and Erection of Stress Grade Lumber and Its Fastenings for Buildings." These specifications increased the working stresses for lumber which was graded for strength and changed the design formulas normally used to agree with this increase. In addition, the safe working loads for all types of fastenings were also increased. However, since this change in specifications applied primarily to buildings and was intended as an emergency measure, it was felt that no revision of *Modern Timber Design* was necessary.

In 1944 the National Lumber Manufacturers Association recommended the adoption of Directive 29 as a permanent design manual and published "National Design Specification for Stress Grade Lumber and Its Fastenings." In effect, this publication is similar to Directive 29 in that the working stresses were given an over-all increase of 20 per cent. This specification is now widely used and has been generally adopted by the practicing engineer. It seemed advisable, therefore, to revise the first edition of *Modern Timber Design* to conform with standard practice.

The revision is quite extensive because the change in working stresses and specifications affected almost every chapter. Since the book was undergoing an over-all revision, an excellent opportunity was provided to add information and data that were found lacking in the first edition. Many users of the book thought that more explanation was needed about the testing methods for wood and about the factors affecting the strength of wood. Because most textbooks on strength of materials do not include the latest information on the relationship between stresses and strains in wood under load, I felt that this material should also be added. Consequently, two new chapters have been added. These are entitled "Mechanical and Related Properties," and "Mechanics of Wood."

I am particularly indebted to the Forest Products Laboratory, the National Lumber Manufacturers Association, and the Timber Engineering Company for the use of data and information contained in their publications.

Howard J. Hansen

Gainesville, Florida
January, 1948

PREFACE TO THE FIRST EDITION

Part of the material in this volume was used by the author for a course in modern timber design given as one of the engineering, science, and management defense training courses at Tulane University. Since then, the original material has been revised, new subjects have been covered, and many illustrative problems have been added. The present volume is used in a comprehensive course in timber design given to first-semester seniors at the Agricultural and Mechanical College of Texas. Since a course in indeterminate structures is offered in the last semester of the senior year, this book does not include the design of this type of wood structure. However, all the basic information needed by the designing engineer is included. The book is intended not only as a textbook but also as a guide and reference for practicing engineers.

Because wood is different from other materials and because it is nonhomogeneous, most of the design data have been derived from test results. It has been the function of the personnel at the Forest Products Laboratory at Madison, Wisconsin, which is a part of the United States Department of Agriculture, to develop the necessary data for scientific design in wood, and most of the available information is a result of their tests and interpretations. Naturally, then, no book on timber design can be written without due acknowledgment to the staff of the Forest Products Laboratory. Generous use has been made of their publications.

I gratefully acknowledge the help given by the National Lumber Manufacturers Association, the Timber Engineering Company, the West Coast Lumbermen's Association, the Southern Pine Association, the Douglas Fir Plywood Association, and the Service Bureau of the American Wood Preservers' Association.

I am also grateful for the criticisms and suggestions offered by Professor C. E. Sandstedt in his review of the contents.

Howard J. Hansen

College Station, Texas
January, 1943

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CHAPTER I

CHARACTERISTICS AND PROPERTIES OF WOOD

1. General. There are more than 180 species of trees grown in the United States that may be considered commercially important. However, the number suitable for structural purposes is relatively small. Nineteen distinct groups have been assigned working stresses; these are ash, beech, birch, chestnut, cypress, elm, fir, gum, hemlock, hickory, larch, maple, oak, pecan, pine, poplar, redwood, spruce, and tupelo. Many of these comprise several species that have individual commercial names and are distinguishable because of their locality of growth and their physical and mechanical properties. Of all the groups, two are used most extensively for structural lumber. These are the southern pines, which grow in the Atlantic and Gulf states and as far north as West Virginia and Kentucky; and the Douglas firs, which grow in the coastal region of Washington, Oregon, and California and in the inland region of Montana, Idaho, Washington, and Oregon.

In using any species in the design and construction of timber structures, it is not necessary for the engineer to know a great deal about the chemical composition of wood. However, an understanding of this subject will often aid him in selecting the proper species under various conditions of use. Of more importance to the designing engineer are the mechanical properties of the different species, the factors affecting their strength, the intelligent use of their assigned working stresses, and the characteristics which make their use in design different from other structural materials.

2. Hardwoods and Softwoods. All species are divided into two general classes, namely, hardwoods and softwoods. However, no definite degree of hardness divides the two groups. Differences in structure, appearance, size, and quality keep the two groups separated. Many of the hardwoods such as oak and hickory are very hard, whereas many of the softwoods are soft; but there are many exceptions such as basswood which is classed as a hardwood but is among the softest of native woods. Longleaf pine, on the other hand, is classed as a softwood but is about as hard as any hardwood. Softwoods are also called "conifers" because most species are cone bearing. The two groups are most accurately differentiated by calling all trees with needle leaves softwoods and all broad-leaved trees hardwoods.

3. Structure. Wood is composed of elongated cells whose framework is cellulose. The cells are cemented together by lignin, and their arrangement in the tree greatly affects the appearance and properties of the different species. The cross section of most trees will show certain features that are common to all trees; Fig. 1 illustrates the important parts that make up the structure of any tree.

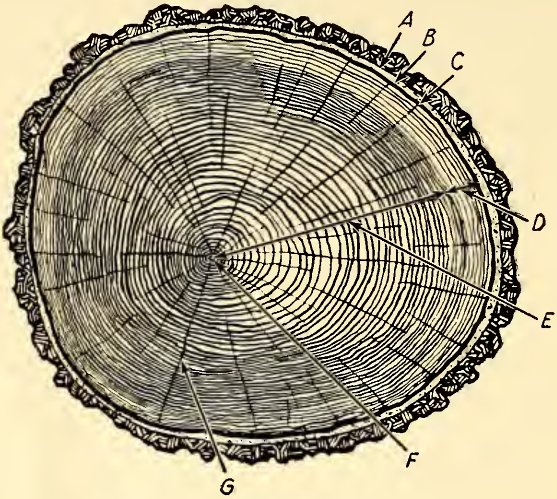

FIG. 1. Typical cross section of a tree.

A is dry, dead tissue called outer bark which serves as a protective coating.

B is the moist soft inner bark which carries food from the leaves to other parts of the tree.

C is a microscopic layer just inside the inner bark which is called the cambium. It is here that new wood and bark cells are formed.

D is the sapwood, which is light in color. Its function is to carry sap from the roots of the tree to the leaves.

E is the heartwood, which is usually dark in color. It is formed by a gradual change in sapwood and is inactive in the tree.

F is the pith. It is here that the new wood growth for twigs takes place.

G represents wood rays which connect the various parts of a tree for the storage and movement of food.

4. Heartwood and Sapwood. Heartwood is formed by a gradual change in sapwood. When wood is used under conditions conducive to decay, a large amount of heartwood in the cross section of a piece is desirable because the heartwood of all species is more durable than the sapwood. However, if the material is to be treated, sapwood is preferable because it absorbs preservatives more readily.

The heartwood of some species is considerably more durable than others. Table 1 may be used as a guide in comparing a number of species.

5. Growth Rings. As shown in Fig. 1, concentric rings start at the center or pith of the tree and continue outward toward the bark. Each ring represents the growth of the tree during one year. This growth is in the cambium, so that the new wood is added just inside this layer and tends to push the bark outward. When trees grow in a variable climate it is possible to distinguish one growth ring from another because the cells formed during a cold season are different from those formed during a warm season.

6. Springwood and Summerwood. Each annual ring is divided into two layers. The inner one, called springwood, is developed during the first part of the growing season. It is composed of large cells with thin walls and is usually lighter in color than the summerwood. The outer layer, called summerwood, consists of smaller cells with thicker walls and is the dark portion of the annual ring. It is heavier and stronger than springwood and has an important effect upon the strength properties of most species.

7. Density. Density is a definite criterion of the strength of softwoods. It is determined by the rate of tree growth and the amount of summerwood present. This means that the strength of a piece of wood is being measured by the amount and distribution of wood substance, which is the material making up the cell walls. The specific gravity of this wood substance is the same for all woods, 1.54, and all woods would be of the same specific gravity throughout were it not for the difference in the arrangement and size of cells and the thickness of the cell walls.

In grading softwood lumber for structural purposes, the number of rings per inch radially and the proportion of summerwood in the cross section of the piece are considered as part of the specifications. Material having a specified minimum number of rings per inch is termed "close-grained," and material that has in addition one-third or more of summerwood is termed "dense."

8. Grain. The wood from trees of rapid growth will have wide annual growth rings and is called coarse-grained. On the other hand wood from slow-growing trees has narrow growth rings and is often called close-grained. Straight grain and cross grain are used to describe wood in which the fibers are parallel to, or at an angle to, the sides of the piece.

Table 1. Durability of Heartwood of Various Species

[Wood Handbook, U. S. Dept. Agr., 1935]

Heartwood durable even when used under conditions that favor decay.	Cedar, Alaska.
	Cedar, eastern red.
	Cedar, northern white.
	Cedar, Port Orford.
	Cedar, southern white.
	Cedar, western red.
	Chestnut.
	Cypress, southern.
	Locust, black.
	Osage-orange.
	Redwood.
	Walnut, black.
	Yew, Pacific.
Heartwood of intermediate durability but nearly as durable as some of the species named in the high-durability group.	Douglas fir (dense).
	Honey locust.
	Oak, white.
	Pine, southern yellow (dense).
Heartwood of intermediate durability.	Douglas fir (unselected).
	Gum, red.
	Larch, western.
	Pine, southern yellow (unselected).
	Tamarack.
Heartwood between the intermediate and the nondurable group.	Ash, commercial white.
	Beech.
	Birch, sweet.
	Birch, yellow.
	Hemlock, eastern.
	Hemlock, western.
	Hickory.
	Maple, sugar.
	Oak, red.
	Spruce, black.
	Spruce, Engelmann.
	Spruce, red.
	Spruce, Sitka.
	Spruce, white.
Heartwood low in durability when used under conditions that favor decay.	Aspen.
	Basswood.
	Cottonwood.
	Fir, commercial white.
	Willow, black.

The slope of grain, expressed as a ratio between a one-inch deviation of the grain from the side of the piece and the distance within which this deviation occurs, is taken into consideration in the specifications for structural timbers because it has a marked effect on the strength of the piece.

Grain, however, usually refers to the appearance of the piece. Lumber sawed in such a manner that the annual rings form an angle of 45° or more with the surface of the piece is called edge grain, vertical grain, or rift sawn in softwoods and quarter sawn in hardwoods. Flat grain or plain sawn refers to lumber that has been sawed approximately tangent to the growth rings, that is, the rings form an angle of less than 45° with the surface of the piece.

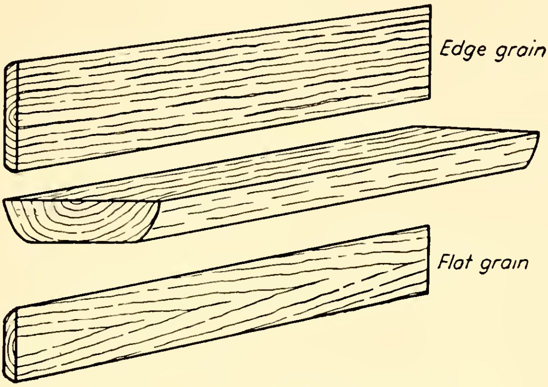

FIG. 2. Flat- and edge-grain lumber.

9. Knots. The many types and classifications of knots depend upon the appearance of the knot on a sawed surface and on whether the knot is the result of a limb that was alive or dead when the tree was cut. The engineer should be concerned with the influence of the knot on the strength of a piece of wood rather than with the type of knot. Knots affect the strength because it is necessary for the grain to deviate from its regular direction in passing around them, and because checking often results in and around them as the piece loses moisture. The influence on strength is determined by the location of the knot and the area that it occupies in the cross section of a piece.

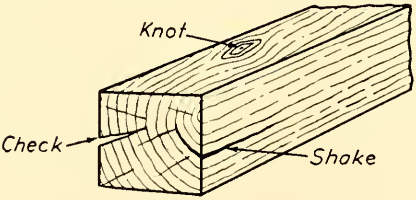

FIG. 3. Knots, checks, and shakes.

The rules for grading structural lumber, which depend upon the working stress assigned to the individual grade and whether or not the piece is to be used as a beam or column, limit the size of knots.

10. Checks and Shakes. A check is a lengthwise separation of the wood across the annual growth rings, and a shake is a separation along the grain between the annual growth rings. A number of checks and shakes in a piece will reduce its working value in shear. The amount and location of checks and shakes are limited in structural timber grades. In some grading rules the unit working stress in horizontal shear is the same for all structural grades, but higher or lower horizontal shear stresses may be secured within certain limits by specifying the working stress that is desired.

11. Moisture Content. Wood contains a considerable amount of free water within its cell walls. After a tree is cut it begins to lose moisture, and the moisture content continues to drop throughout the entire manufacturing process. This moisture content is defined as the weight of water contained in wood expressed as a percentage of the weight of the oven-dry wood. When put into use, wood will continue to dry until it is in equilibrium with the surrounding atmosphere. By equilibrium moisture content is meant the ultimate moisture content depending upon the temperature and the relative humidity in the atmosphere, that wood will attain. Chart 1 shows the average relationship between the moisture content of wood and the relative humidity of the surrounding atmosphere at three temperatures.

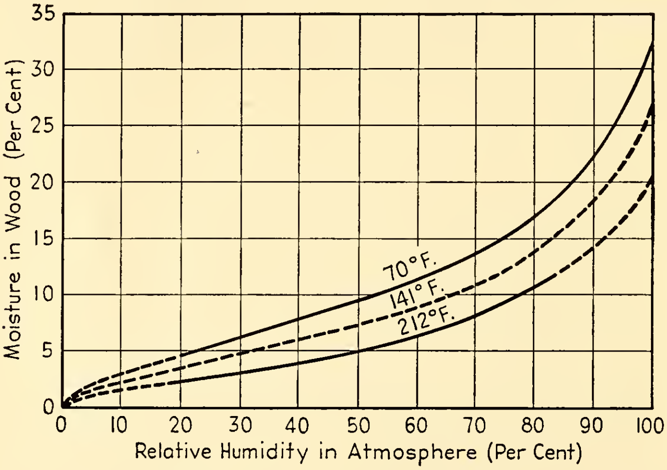

Chart 1. Humidity-moisture-content relationship at three temperatures. (*Wood Handbook, U. S. Dept. Agr., 1935.*)

Wood is said to have reached the fiber saturation point when all the free water in the cells is evaporated and the cell walls are still saturated. This point is generally reached when the moisture content is between 24 and 30 per cent. As wood dries beyond the fiber saturation point it increases in strength. However, such properties as toughness or shock resistance decrease because wood in a dry condition will not bend as much as wood in a green condition. In commercial structural grades the increase in strength due to drying is to a large extent offset by the influence of defects that develop during seasoning.

12. Shrinkage. Shrinkage takes place after the fiber saturation point has been reached. Accompanied by a reduction in moisture content is a reduction in the size of a piece, for as wood dries it also shrinks. Wood shrinks most tangentially or in the direction of the annual growth rings and about one-half to two-thirds as much radially or across the annual rings. The longitudinal shrinkage in most woods is negligible. Tests at the Forest Products Laboratory have established average shrinkage values for all species. Chart 2, which shows typical moisture-shrinkage curves for Douglas fir and southern pine, may be used for estimating the amount of change in dimension that will take place with a change in moisture content.

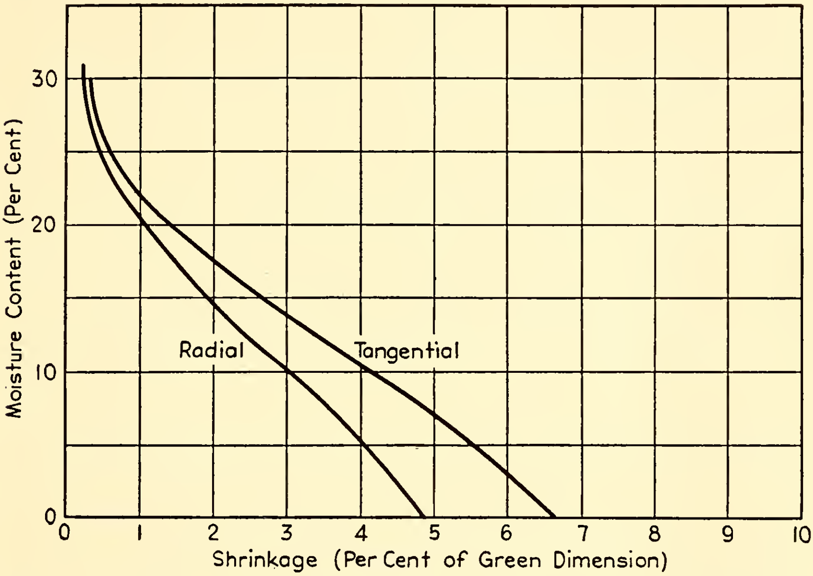

Chart 2. Average moisture-shrinkage curves for southern pine and Douglas fir. (*Wood Handbook, U. S. Dept. Agr., 1935.*)

Naturally, the engineer should attempt to secure lumber that is seasoned to its ultimate moisture content. However, it is seldom possible to obtain large timbers fully seasoned, and therefore a certain amount of shrinkage is to be expected and provided for in the design. Methods of design and construction that provide vertical bearings across the grain should be avoided. For instance, the use of joists placed directly on top of a girder increases the vertical height of wood used and results in increased shrinkage. If the joists bear on ledger strips fastened to the sides of a girder, there is a reduction in vertical height and less shrinkage. In the construction of mill buildings and similar structures this same idea can be carried out to eliminate part of the shrinkage by using metal post caps to separate the upper column from the lower column. This means that the girder is not being used as a bearing for the upper column. The use of a cast-iron pintle, bearing on a metal post cap, will produce the same effect and will also allow the girder to bear directly on the lower column.

Structural joints fabricated with green wood and loaded before seasoning takes place should be inspected at regular intervals while the timber is undergoing a reduction in moisture content. If nails become loose, simply resetting the old nails is not sufficient; the joint should be reinforced with additional nails. The design details for bolted joints and for joints employing connectors, given in Chapter V, provide reductions in the safe design loads for different conditions of seasoning, but these joints should be inspected for serious checks that might weaken them and for loose bolts.

13. Sizes. A table of sizes for the various items of lumber manufactured is included in the Appendix. For material 2, 2½, 3, and 4 in. in nominal thickness, the actual thickness or dressed size becomes 1⅝, 2⅛, 2⅝, and 3⅝ in. respectively; the actual width is ⅜ in. off the nominal width through 7 in. and ½ in. off the nominal width above that dimension. For material 5 in. thick and thicker by 5 in. wide and wider, the actual thickness and width are ½ in. off the nominal. Calculations for areas and moments of inertia are based on actual sizes. However, lumber is sold on the basis of the contents of the nominal size expressed in terms of board feet. A board foot is the contents of a volume 12 by 12 by 1 in.

CHAPTER II

MECHANICAL AND RELATED PROPERTIES

14. Strength Properties. The structure of wood makes it unlike any other structural material, and no two pieces of wood have exactly the same strength properties. Consequently, the development of the mechanical properties for important species required thousands of tests in order to obtain true average values. Several hundred thousand tests have been performed at the Forest Products Laboratory on small clear specimens from 164 species. Table 2 gives the strength properties of the commercially important species; information on additional species appears in Technical Bulletin 479, published by the United States Department of Agriculture.

The values in Table 2 are primarily for the comparison of species in the form of clear wood and should not be used in design.

TESTING METHODS

The mechanical properties given in Table 2 are the results of tests on specimens 2 by 2 in. in cross section and of various lengths, depending on the type of test. Only clear specimens free from knots, cross grain, shakes, and checks were tested.

15. Static Bending. In the static bending test the specimen is 2 by 2 in. in cross section and 30 in. long, supported on roller bearings which rest on knife edges 28 in. apart. The load is applied at the center of the span through a hard maple block $3\frac{13}{16}$ in. wide with a curvature of 3 in. radius over the central $2\frac{1}{8}$ in. of arc and a curvature of 2-in. radius on each side (Fig. 4). The beam is placed with the annual rings horizontal and a constant rate of deflection (0.1 in. per minute) is maintained until the beam fails.

The fiber stress at the proportional limit is the stress at which the load ceases to increase in direct proportion to the deflection.

The modulus of rupture is the stress obtained by using the maximum load, instead of the load at the proportional limit, in the formula $f = Mc/I$.

Table 2. Strength Properties of Commercially Important Species

[Compiled from *Tech. Bull.* 479, U. S. Dept. Agr., 1935]

Commercial and Botanical Name of Species	Moisture Content	Specific Gravity	Static Bending					Impact Bending		Compression Parallel to Grain		Compression Perpendicular to Grain—Fiber Stress at Proportional Limit	Shear Parallel to Grain—Maximum Shearing Strength	Hardness	
			Fiber Stress at Proportional Limit	Modulus of		Work to		Fiber Stress at Proportional Limit	Height of Drop Causing Complete Failure (50-Lb. Hammer)	Fiber Stress at Proportional Limit	Maximum Crushing Strength			Load Required to Embed a 0.444-In. Ball to One-Half Its Diameter	
				Rupture	Elasticity	Proportional Limit	Maximum Load							End	Side
	per cent		*lb. per sq. in.*	*lb. per sq. in.*	*1,000 lb. per sq. in.*	*in.-lb. per cu. in.*	*in.-lb. per cu. in.*	*lb. per sq. in.*	*in.*	*lb. per sq. in.*	*lb. per sq. in.*	*lb. per sq. in.*	*lb. per sq. in.*	*lb.*	*lb.*
Alder, red (*Alnus rubra*)	98	0.37	3,800	6,500	1,170	0.70	8.0	8,000	22	2,620	2,960	310	770	550	440
	12	0.41	6,900	9.800	1,380	1.85	8.4	11,600	20	4,530	5,820	540	1,080	980	590
Ash, black (*Fraxinus nigra*)	85	0.45	2,600	6,000	1,040	0.41	12.1	.....	33	1,690	2,300	430	860	590	....
	12	0.49	7,200	12,600	1,600	1.57	14.9	.....	35	4,520	5,970	940	1,150	1,150	....
Ash, commercial white (*Fraxinus* sp.)	43	0.54	5,300	9,500	1,400	1.14	14.7	12,800	37	3,360	4,060	860	1,350	1,010	940
	12	0.58	8,900	14,600	1,680	2.68	15.6	17,000	40	5,580	7,280	1,510	1,920	1,680	1,260
Ash, Oregon (*Fraxinus oregona*)	48	0.50	4,200	7,600	1,130	0.92	12.2	8,900	39	2,760	3,510	650	1,190	850	790
	12	0.55	7,000	12,700	1,360	2.08	14.4	13,300	33	4,100	6,040	1,540	1,790	1,430	1,160
Aspen (*Populus tremuloides*)	94	0.35	3,200	5,100	860	0.69	6.4	7,000	22	1,670	2,140	220	660	280	300
	12	0.38	5,600	8,400	1,180	1.53	7.6	9,000	21	3,040	4,250	460	850	510	350
Basswood (*Tilia glabra*)	105	0.32	2,700	5,000	1,040	0.40	5.3	6,300	16	1,690	2,220	210	600	290	250
	12	0.37	5,900	8,700	1,460	1.37	7.2	9,800	16	3,800	4,730	450	990	520	410
Beech (*Fagus grandifolia*)	54	0.56	4,300	8,600	1,380	0.85	11.9	11,500	43	2,550	3,550	670	1,290	970	850
	12	0.64	8,700	14,900	1,720	2.63	15.1	16,000	41	4,880	7,300	1,250	2,010	1,590	1,300
Birch (*Betula* sp.)	62	0.57	4,400	8,700	1,560	0.79	15.9	11,100	48	2,640	3,510	550	1,160	910	850
	12	0.63	10,100	16,700	2,070	2.83	19.8	20,000	52	6,200	8,310	1,250	2,020	1,660	1,340
Birch, paper (*Betula papyrifera*)	65	0.48	3,000	6,400	1,170	0.45	16.2	8,000	49	1,640	2,360	340	840	470	560
	12	0.55	6,900	12,300	1,590	1.80	16.0	12,400	34	3,610	5,690	740	1,210	890	910
Butternut (*Juglans cinerea*)	104	0.36	2,900	5,400	970	0.52	8.2	7,300	24	2,020	2,420	270	760	410	390
	12	0.38	5,700	8,100	1,180	1.59	8.2	11,200	24	4,200	5,110	570	1,170	570	490
Cedar, Alaska (*Chamaecyparis nootkatensis*)	38	0.42	3,800	6,400	1,140	0.77	9.2	9,100	27	2,500	3,050	430	840	540	440
	12	0.44	7,100	11,100	1,420	2.06	10.4	12,200	29	5,210	6,310	770	1,130	790	580
Cedar, eastern red (*Juniperus virginiana*)	35	0.44	3,400	7,000	650	1.08	15.0	7,000	35	2,540	3,570	860	1,010	760	650
	12	0.47	3,800	8,800	880	1.01	8.3	8,500	22	....	6,020	1,140	....	900	....
Cedar, incense (*Libocedrus decurrens*)	108	0.35	3,900	6,200	840	0.94	6.4	7,300	17	2,940	3,150	460	830	570	390
	12	...	5,900	8,000	1,040	1.67	5.4	9,600	17	4,760	5,200	730	880	830	470

Cedar, northern white (*Thuja occidentalis*)	55	0.29	2,600	4,200	640	0.60	5.7	5,300	15	1,490	1,990	290	620	320	230
	12	0.31	4,909	6,500	800	1.72	4.8	7,100	12	2,630	3,960	380	850	450	320
Cedar, Port Orford (*Chamaecyparis lawsoniana*)	43	0.40	4,000	6,200	1,420	0.65	7.4	9,200	22	2,770	3,130	350	830	460	400
	12	0.42	7,700	11,300	1,730	1.97	9.1	13,500	28	5,890	6,470	760	1,080	730	560
Cedar, southern white (*Chamaecyparis thyoides*)	35	0.31	2,500	4,700	750	0.51	5.9	6,000	18	1,660	2,390	300	690	400	290
	12	0.32	4,800	6,800	930	1.46	4.1	7,600	13	2,740	4,700	500	800	520	350
Cedar, western red (*Thuja plicata*)	37	0.31	3,200	5,100	920	0.63	5.0	6,900	17	2,470	2,750	340	710	430	270
	12	0.33	5,300	7,700	1,120	1.44	5.8	8,600	17	4,360	5,020	610	860	660	350
Cherry, black (*Prunus serotina*)	55	0.47	4,200	8,000	1,310	0.80	12.8	10,200	33	2,940	3,540	440	1,130	750	660
	12	0.50	9,000	12,300	1,490	3.11	11.4	13,600	29	5,960	7,110	850	1,700	1,470	950
Chestnut (*Castanea dentata*)	122	0.40	3,100	5,600	930	0.59	7.0	7,900	24	2,080	2,470	380	800	530	420
	12	0.43	6,100	8,600	1,230	1.78	6.5	10,700	19	3,780	5,320	760	1,080	720	540
Cottonwood, eastern (*Populus deltoides*)	111	0.37	2,900	5,300	1,010	0.49	7.3	7,200	21	1,740	2,280	240	680	380	340
	12	0.40	5,700	8,500	1,370	1.39	7.4	7,300	20	3,490	4,910	470	930	580	430
Cottonwood, northern black (*Populus trichocarpa hastata*)	132	0.32	2,900	4,800	1,070	0.44	5.0	6,800	20	1,760	2,160	200	600	280	250
	12	0.35	5,300	8,300	1,260	1.25	6.7	9,800	22	3,270	4,420	370	1,020	540	350
Cypress, southern (*Taxodium distichum*)	91	0.42	4,200	6,600	1,180	0.91	6.6	8,800	25	3,100	3,580	500	810	440	390
	12	0.46	7,200	10,600	1,440	2.15	8.2	10,400	24	4,740	6,360	900	1,000	660	510
Douglas fir (coast region) (*Pseudotsuga taxifolia*)	36	0.45	4,800	7,600	1,550	0.85	6.8	9,800	24	3,410	3,890	510	930	510	480
	12	0.48	8,100	11,700	1,920	1.96	8.6	12,700	30	6,450	7,420	910	1,140	760	670
Douglas fir ("Inland Empire" region) (*Pseudotsuga taxifolia*)	42	0.41	3,600	6,800	1,340	0.55	6.9	8,700	22	2,460	3,240	500	870	530	470
	12	0.44	7,400	11,300	1,610	1.91	8.6	11,800	27	5,520	6,700	950	1,190	720	630
Douglas fir (Rocky Mountain region) (*Pseudotsuga taxifolia*)	38	0.40	3,600	6,400	1,180	0.65	6.8	9,100	20	2,540	3,000	450	880	450	400
	12	0.43	6,300	9,600	1,400	1.60	6.4	12,100	26	4,660	6,060	820	1,070	740	630
Elm, American (*Ulmus americana*)	89	0.46	3,900	7,200	1,110	0.81	11.8	.....	38	1,920	2,910	440	1,000	680	....
	12	0.50	7,600	11,800	1,340	2.53	13.0	.....	39	4,030	5,520	850	1,510	1,110	....
Elm, rock (*Ulmus racemosa*)	48	0.57	4,600	9,500	1,190	1.05	19.8	.....	54	2,970	3,780	750	1,270	980	....
	12	0.63	8,000	14,800	1,540	2.45	19.2	.....	56	4,700	7,050	1,520	1,920	1,510	....
Elm, slippery (*Ulmus fulva*)	85	0.48	4,000	8,000	1,230	0.82	15.4	9,200	47	2,790	3,320	510	1,110	750	660
	12	0.53	7,700	13,000	1,490	2.35	16.9	15,300	45	4,760	6,360	1,010	1,630	1,120	860
Fir, balsam (*Abies balsamea*)	117	0.34	3,000	4,900	960	0.52	4.7	6,900	16	2,080	2,400	210	610	290	290
	12	0.36	5,200	7,600	1,230	1.23	5.1	7,800	20	3,970	4,530	380	710	510	400
Fir, commercial white (*Abies* sp.)	108	0.36	3,800	5,800	1,120	0.75	5.3	8,300	22	2,470	2,810	360	750	390	340
	12	0.38	6,300	9,300	1,470	1.55	7.0	11,200	20	3,870	5,380	610	930	710	460
Gum, black (*Nyssa sylvatica*)	55	0.46	4,000	7,000	1,030	0.91	8.0	9,800	30	2,490	3,040	600	1,100	790	640
	12	0.50	7,300	9,600	1,200	2.54	6.2	14,500	22	3,470	5,520	1,150	1,340	1,240	810
Gum, red (*Liquidambar styraciflua*)	81	0.44	3,700	6,800	1,150	0.81	9.4	10,000	33	2,230	2,840	460	1,070	630	520
	12	0.49	8,100	11,900	1,490	2.57	11.3	16,800	32	4,700	5,800	860	1,610	950	690
Gum, tupelo (*Nyssa aquatica*)	97	0.46	4,200	7,300	1,050	0.98	8.3	9,000	30	2,690	3,370	590	1,190	800	710
	12	0.50	7,200	9,600	1,260	2.41	6.9	12,500	23	4,280	5,920	1,070	1,590	1,200	880
Hackberry (*Celtis occidentalis*)	65	0.49	2,900	6,500	950	0.58	14.5	7,900	48	2,070	2,650	490	1,070	760	700
	12	0.53	5,900	11,000	1,190	1.72	12.8	13,700	43	3,710	5,440	1,100	1,590	1,110	880
Hemlock, eastern (*Tsuga canadensis*)	111	0.38	3,800	6,400	1,070	0.76	6.7	7,900	21	2,600	3,080	440	850	500	400
	12	0.40	6,100	8.900	1,200	1.79	6.8	10,700	21	4,020	5,410	800	1,060	810	500
Hemlock, western (*Tsuga heterophylla*)	74	0.38	3,400	6,100	1,220	0.57	6.8	8,100	22	2,480	2,990	390	810	520	430
	12	0.42	6,800	10,100	1,490	1.82	7.5	12,400	26	5,340	6,210	680	1,170	940	580
Hickory, pecan (*Hicoria* sp.)	68	0.59	5,300	9,900	1,380	1.18	19.3	14,200	60	3,810	4,320	980	1,260	1,274	1,308
	12	0.65	9,100	16,300	1,780	2.61	18.8	20,900	57	6,360	8,280	2,040	1,770	1,930	1,820

Table 2. Strength Properties of Commercially Important Species—*Continued*

[Compiled from *Tech. Bull.* 479, U. S. Dept. Agr., 1935]

Commercial and Botanical Name of Species	Moisture Content	Specific Gravity	Static Bending					Impact Bending		Compression Parallel to Grain		Compression Perpendicular to Grain—Fiber Stress at Proportional Limit	Shear Parallel to Grain—Maximum Shearing Strength	Hardness	
			Fiber Stress at Proportional Limit	Modulus of		Work to		Fiber Stress at Proportional Limit	Height of Drop Causing Complete Failure (50-Lb. Hammer)	Fiber Stress at Proportional Limit	Maximum Crushing Strength			Load Required to Embed a 0.444-In. Ball to One-Half Its Diameter	
				Rupture	Elasticity	Proportional Limit	Maximum Load							End	Side
	per cent		*lb. per sq. in.*	*lb. per sq. in.*	*1,000 lb. per sq. in.*	*in.-lb. per cu. in.*	*in.-lb. per cu. in.*	*lb. per sq. in.*	*in.*	*lb. per sq. in.*	*lb. per sq. in.*	*lb. per sq. in.*	*lb. per sq. in.*	*lb.*	*lb.*
Hickory, true (*Hicoria* sp.)	57	0.65	6,100	11,300	1,570	1.34	28.9	15,700	88	3,650	4,570	1,080	1,360	....	....
	12	0.73	10,900	19,700	2,180	3.07	27.2	22,900	75	....	8,970	2,310	2,140	....	....
Honey locust (*Gleditsia triacanthos*)	63	0.60	5,600	10,200	1,290	1.40	12.6	11,800	47	3,320	4,420	1,420	1,660	1,440	1,390
	12	...	8,800	14,700	1,630	2.74	13.3	15,400	47	5,250	7,500	2,280	2,250	1,860	1,580
Larch, western (*Larix occidentalis*)	58	0.48	4,600	7,500	1,350	1.01	7.1	9,400	24	3,250	3,800	560	920	470	450
	12	0.52	7,900	11,900	1,710	2.46	8.0	15,100	32	5,950	7,490	1,080	1,360	1,110	760
Locust, black (*Robinia pseudoacacia*)	40	0.66	8,800	13,800	1,850	2.36	15.4	18,300	44	6,120	6,800	1,430	1,760	1,640	1,570
	12	0.69	12,800	19,400	2,050	4.62	18.4	21,100	57	6,800	10,180	2,260	2,480	1,580	1,700
Magnolia, cucumber (*Magnolia acuminata*)	80	0.44	4,200	7,400	1,560	0.66	10.0	9,300	30	2,810	3,140	410	990	600	520
	12	0.48	8,000	12,300	1,820	1.98	12.2	14,700	35	4,840	6,310	710	1,340	950	700
Magnolia, evergreen (*Magnolia grandiflora*)	105	0.46	3,600	6,800	1,110	0.67	15.4	8,800	54	2,160	2,700	570	1,040	780	740
	12	0.50	6,800	11,200	1,400	1.90	12.8	13,600	29	3,420	5,460	1,060	1,530	1,280	1,020
Maple, bigleaf (*Acer macrophyllum*)	72	0.44	4,400	7,400	1,100	1.02	8.7	8,500	23	2,510	3,240	550	1,110	760	620
	12	0.48	6,600	10,700	1,450	1.66	7.8	.....	28	4,790	5,950	930	1,730	1,330	850
Maple, black (*Acer nigrum*)	65	0.52	4,100	7,900	1,330	0.70	12.8	10,200	48	2,800	3,270	740	1,130	940	840
	12	0.57	8,300	13,300	1,620	2.39	12.5	13,500	40	4,600	6,680	1,250	1,820	1,700	1,180
Maple, red (*Acer rubrum*)	63	0.49	3,800	7,700	1,390	0.71	11.4	.....	32	2,360	3,280	500	1,150	780	....
	12	0.54	8,700	13,400	1,640	2.84	12.5	.....	32	4,650	6,540	1,240	1,850	1,430	....
Maple, silver (*Acer saccharinum*)	66	0.44	3,100	5,800	940	0.61	11.0	6,800	29	1,930	2,490	460	1,050	670	590
	12	0.47	6,200	8,900	1,140	1.90	8.3	12,400	25	4,360	5,220	910	1,480	1,140	700
Maple, sugar (*Acer saccharum*)	58	0.56	5,100	9,400	1,550	1.03	13.3	12,200	40	2,850	4,020	800	1,460	1,070	970
	12	0.63	9,500	15,800	1,830	2.76	16.5	20,600	39	5,390	7,830	1,810	2,330	1,840	1,450
Oak, red (*Quercus* sp.)	80	0.57	4,400	8,500	1,360	0.85	12.6	10,800	43	2,590	3,520	800	1,220	1,050	1,030
	12	0.63	8,400	14,400	1,810	2.30	15.0	17,000	43	4,610	6,920	1,260	1,830	1,490	1,300
Oak, white (*Quercus* sp.)	70	0.59	4,700	8,100	1,200	1.08	11.3	10,900	42	2,940	3,520	850	1,270	1,110	1,070
	12	0.67	7,900	13,900	1,620	2.31	13.3	17,400	39	4,350	7,040	1,410	1,890	1,420	1,330

Pine, lodgepole (*Pinus contorta*)	65	0.38	3,000	5,500	1,080	0.49	5.6	7,200	20	2,110	2,610	310	680	320	330
	12	0.41	6,700	9,400	1,340	1.97	6.8	9,600	20	4,310	5,370	750	880	530	480
Pine, northern white (*Pinus strobus*)	68	0.34	3,100	5,000	1,020	0.54	5.2	6,700	17	2,060	2,490	290	660	310	310
	12	0.36	6,000	8,800	1,280	1.59	6.7	9,500	19	3.680	4,840	550	860	500	400
Pine, Norway (*Pinus resinosa*)	54	0.44	3,700	6,400	1,380	0.59	5.8	7,500	28	2,410	3,080	360	780	360	340
	12	0.48	9,400	12,500	1,800	2.78	10.0	15,900	25	5,330	7,340	830	1,230	670	580
Pine, ponderosa (*Pinus ponderosa*)	91	0.38	3,100	5,000	970	0.59	5.1	6,800	20	2,070	2,400	360	680	300	310
	12	0.40	6,300	9,200	1,260	1.85	6.6	9,800	17	4,060	5,270	740	1,160	550	450
Pines, southern yellow:															
Loblolly (*Pinus taeda*)	81	0.47	4,100	7,300	1,410	0.68	8.2	8,900	30	2,550	3,490	480	850	420	450
	12	0.51	7,800	12,800	1,800	1.92	10.4	12,100	30	4,820	7,080	980	1,370	750	690
Longleaf (*Pinus palustris*)	63	0.54	5,200	8,700	1,600	0.95	8.9	10,100	35	3,430	4,300	590	1,040	550	590
	12	0.58	9,300	14,700	1,990	2.44	11.8	15,400	34	6,150	8,440	1,190	1,500	920	870
Shortleaf (*Pinus echinata*)	81	0.46	3,900	7,300	1,390	0.63	8.2	8,600	30	2,500	3,430	440	850	410	440
	12	0.51	7,700	12,800	1,760	1.93	11.0	13,600	33	5,090	7,070	1,000	1,310	750	690
Pine, sugar (*Pinus lambertiana*)	137	0.35	3,400	5,100	940	0.70	5.4	7,400	17	2,330	2,530	350	680	320	310
	12	0.36	5,700	8,000	1,200	1.53	5.5	10,700	18	4,140	4,770	590	1,050	530	380
Pine, western white (*Pinus monticola*)	54	0.36	3,400	5,200	1,170	0.56	5.0	7,600	19	2,430	2,650	290	640	310	310
	12	0.38	6,200	9,500	1,510	1.47	8.8	11,900	23	4,480	5,620	540	....	440	370
Poplar, yellow (*Liriodendron tulipifera*)	64	0.38	3,400	5,400	1,090	0.62	5.4	8,600	18	1,930	2,420	330	740	390	340
	12	0.40	6,100	9,200	1,500	1.43	6.8	13,500	20	3,550	5,290	580	1,100	560	450
Redwood (virgin) (*Sequoia sempervirens*)	112	0.38	4,800	7,500	1,180	1.18	7.4	8,900	21	3,700	4,200	520	800	570	410
	12	0.40	6,900	10,000	1,340	2.04	6.9	10,200	19	4,560	6,150	860	940	790	480
Spruce, eastern (*Picea* sp.)	46	0.38	3,300	5,600	1,110	0.57	6.5	7,000	21	2,120	2,600	290	710	390	340
	12	0.40	6,500	10,100	1,440	1.68	8.4	11,400	22	4,160	5,590	590	1,070	630	490
Spruce, Engelmann (*Picea engelmannii*)	100	0.31	2,500	4,200	830	0.43	4.9	5,800	14	1,680	1,980	290	590	250	240
	12	0.33	6,000	8,500	1,160	1.64	5.6	9,000	15	3,580	4,580	640	1,010	450	310
Spruce, Sitka (*Picea sitchensis*)	42	0.37	3,300	5,700	1,230	0.53	6.3	8,400	24	2,240	2,670	340	760	430	350
	12	0.40	6,700	10,200	1,570	1.62	9.4	11,400	25	4,780	5,610	710	1,150	760	510
Sugarberry (*Celtis laevigata*)	62	0.47	3,200	6,600	810	0.78	12.0	8,200	33	1,990	2,800	580	1,050	840	740
	12	0.51	6,200	9,900	1,140	2.18	11.2	11,600	36	3,970	5,620	1,240	1,280	1,280	960
Sycamore (*Platanus occidentalis*)	83	0.46	3,300	6,500	1,060	0.60	7.5	8,800	26	2,400	2,920	450	1,000	700	610
	12	0.49	6,400	10,000	1,420	1.66	8.5	10,500	26	3,710	5,380	860	1,470	920	770
Tamarack (*Larix laricina*)	52	0.49	4,200	7,200	1,240	0.84	7.2	7,800	28	2,930	3,480	480	860	400	380
	12	0.53	8,000	11,600	1,640	2.19	7.1	12,500	23	4,780	7,160	990	1,280	670	590
Walnut, black (*Juglans nigra*)	81	0.51	5,400	9,500	1,420	1.16	14.6	11,900	37	3,520	4,300	600	1,220	960	900
	12	0.55	10,500	14,600	1,680	3.70	10.7	16,400	34	5,780	7,580	1,250	1,370	1,050	1,010

The modulus of elasticity is a measure of the stiffness or resistance to deflection and is calculated by using the load and deflection at the proportional limit in the usual deflection formula.

The work to the proportional limit is a measure of the work that a beam is able to resist or the shock that it can absorb without being stressed beyond the proportional limit. It is the product of the average load up to the proportional limit times the deflection at the proportional limit and is represented by the area under the load deflection curve up to the proportional limit.

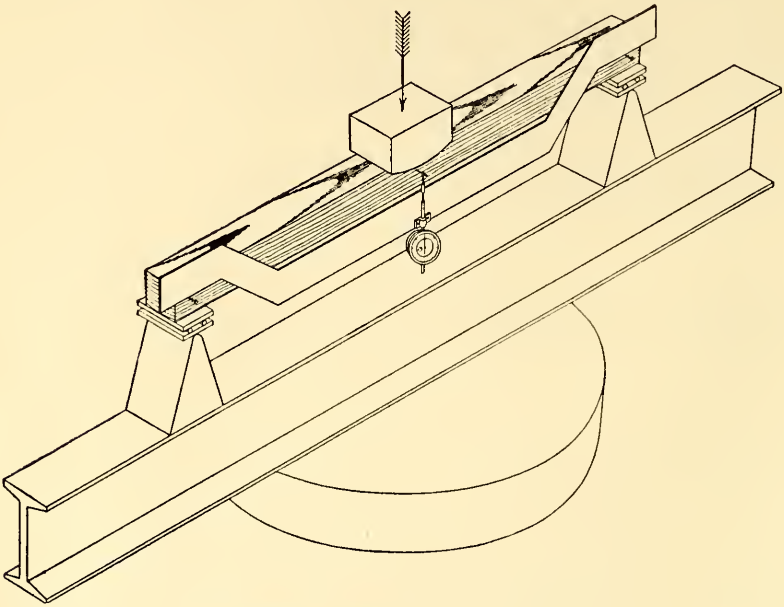

Fig. 4. Method of conducting static bending test.

Work to maximum load is a measure of the combined strength and toughness of a beam under bending stresses and is represented by the area under the load-deflection curve up to the deflection at which the maximum load is sustained.

16. Impact Bending. The test for impact bending is made to determine the resistance of a beam to suddenly applied loads. The specimen is 2 by 2 in. in cross section and 30 in. long with a clear span of 28 in. A 50-lb. hammer falling between vertical guides is dropped on the center of the beam from a height of 1 in. on up to 10 in. in increments of 1 in. Then the height of drop is increased to 2 in. each time until the beam fails. A stylus attached to the hammer records the deflection at each blow and the position of the beam when the hammer comes to rest after rebounding.

The fiber stress at the proportional limit is the stress at which the load-deformation curve ceases to be a straight line. The fiber stress at the proportional limit in impact bending is approximately double the fiber stress at the proportional limit in static bending, which indicates that a small wooden beam, for suddenly applied loads, bends twice as far to the proportional limit as when the load is applied slowly.

The height of drop is the maximum drop of the hammer causing complete failure of the beam or causing a 6-in. deflection.

17. Compression Parallel to the Grain. The test specimen consists of a 2 by 2 by 8 in. block compressed in the direction of its length at a constant rate (0.024 in. per minute). The load is applied through a spherical bearing block, and the load and deformation in a 6-in. gage length are recorded until the proportional limit is passed. The load at failure is then obtained.

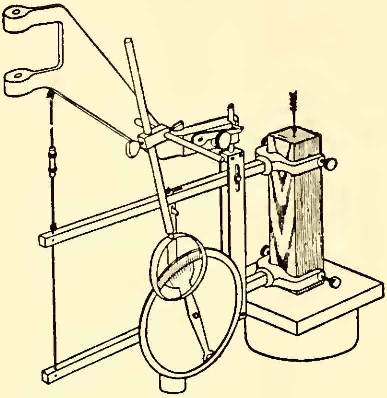

Fig. 5. Method of conducting compression parallel to grain test.

The stress at the proportional limit is the stress at the point beyond which the compression increases more rapidly than the load. It is calculated by dividing the load at the proportional limit by the original cross-sectional area.

The maximum crushing strength is computed by dividing the maximum load that the short block will sustain by the cross-sectional area.

18. Compression Perpendicular to the Grain. The test specimen is 2 by 2 by 6 in. long. The pressure is applied through an iron plate 2 in. wide covering one-third the surface of the specimen. The growth rings of the specimen are placed vertical. The rate of loading is 0.024 in. per minute and readings of the load and the compression are recorded until a compression of 0.1 in. is reached.

The stress at the proportional limit is the stress based on the load at the proportional limit divided by the area under compression.

19. Shear Parallel to the Grain. In this test a shearing force is applied to a 2 by 2 in. lip projecting ¾ in. from the side of a block 2½ in. long. The block is placed in a special tool having a plate that is placed on the lip and the force applied to move the plate at a rate of 0.015 in. per minute.

The maximum shearing strength is the load required to shear off the lip divided by the shearing area.

20. Hardness. The hardness test is applied to the end, radial, and tangential surfaces of the wood specimen and consists of embedding a 0.444-in. ball at a rate of 0.25 in. per minute. The hardness is measured by the load required to embed the ball to one-half its diameter in the wood.

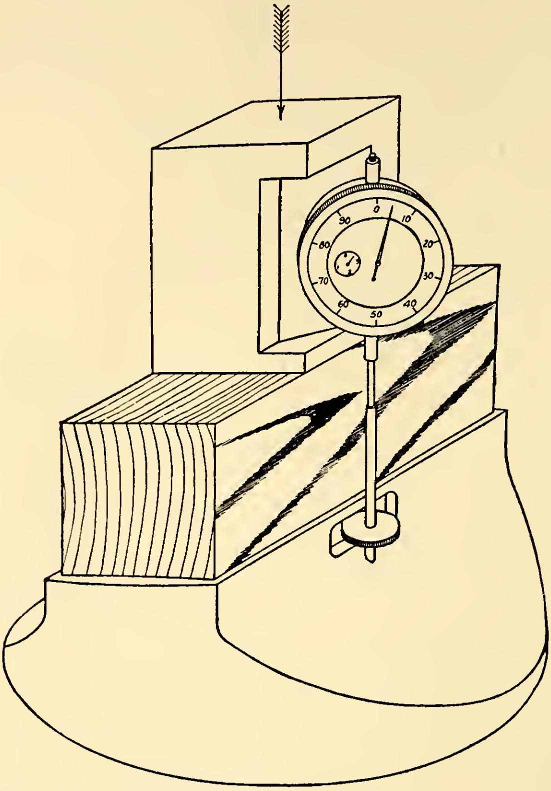

Fig. 6. Method of conducting compression perpendicular to grain test.

21. Tension Perpendicular to the Grain. The resistance of wood across the grain to slowly applied loads is determined by testing a specimen 2 by 2 in. in cross section and 2½ in. long with a length at mid-height of 1 in. Special testing grips are required as shown in Fig. 8. The movable head of the testing machine moves at the rate of 0.25 in. per minute.

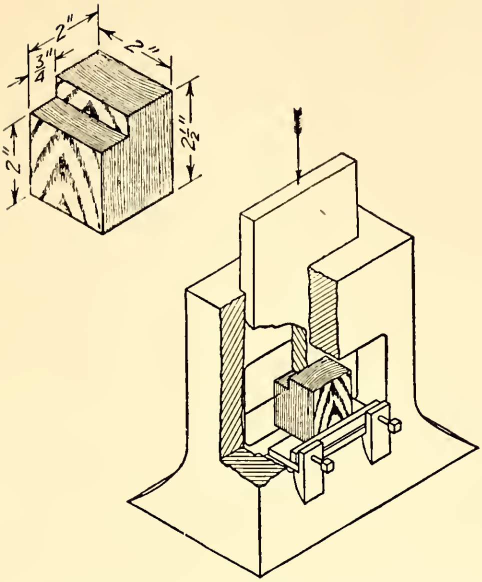

Fig. 7. Method of conducting shear parallel to grain test.

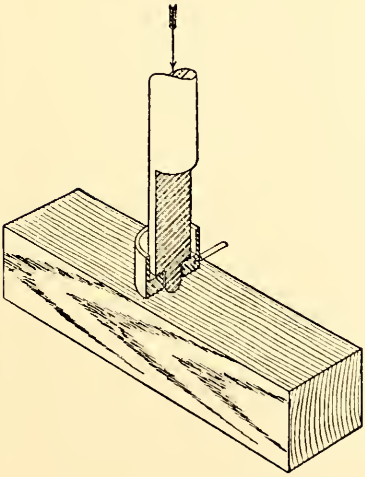

Fig. 8. Method of conducting hardness test.

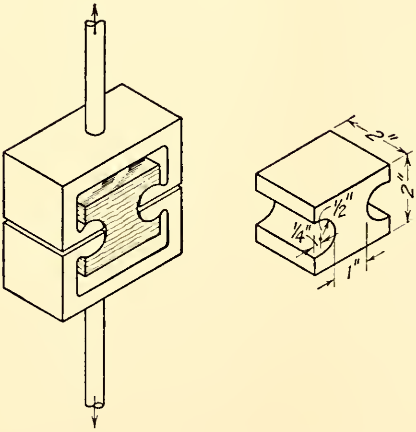

Fig. 9. Method of conducting tension perpendicular to grain test.

The maximum tensile strength is the maximum load divided by the minimum area over which it was distributed. From the type of test it can be seen that the load is not uniformly distributed over the area, and the calculated values do not represent a true tensile stress; but they are useful for comparing the resistance of species to loads acting across the grain.

22. Tension Parallel to the Grain. Very little information is available on the tensile strength of wood along the grain. Great care is necessary in preparing test specimens, and only a small number of tests have been performed. The results of these tests are given in Table 3.

Table 3. Ultimate Tensile Strength Parallel to the Grain of Various Species

[*Tech. Bull.* 479, U. S. Dept. Agr.]

Species	Green				Air-Dry			
	Moisture Content	Tests	Specific Gravity [1]	Ultimate Tensile Strength	Moisture Content	Tests	Specific Gravity [1]	Ultimate Tensile Strength
	per cent	*number*		*lb. per sq. in.*	*per cent*	*number*		*lb. per sq. in.*
Ash, white	...	1	0.535	16,150	...	..	....	.....
Beech	53	1	0.569	12,530	...	..	....	.....
Cedar:								
Port Orford	34	34	0.393	11,380	...	..	....	.....
Western red	40	10	0.300	6,200	8.8	7	0.323	7,130
Cypress, southern	78	15	0.424	8,720	...	..	....	.....
Douglas fir:								
Coast	24	48	0.425	12,980	11.1	8	0.444	13,830
"Inland Empire"	30	9	0.409	9,380	10.2	1	0.474	14,880
Fir:								
Noble	29	11	0.353	14,750	10.2	9	0.370	13,020
California red	168	14	0.373	9,040	10.1	10	0.385	10,750
White	48	9	0.367	8,030	10.7	6	0.382	10,450
Hemlock, western	67	20	0.380	9,860	10.9	14	0.400	9.820
Maple, sugar	48	5	0.577	15,660	...	..	....	.....
Oak, pin	80	3	0.578	16,260	...	..	....	.....
Pine:								
Loblolly	47	2	0.446	11,570	11.6	1	0.484	15,050
Ponderosa	69	11	0.364	8,320	...	..	....	.....
Poplar, balsam	106	3	0.298	7,940	10.4	2	0.351	12,160
Redwood	104	29	0.377	9,780	10.7	33	0.401	10,920
Spruce:								
Eastern [2]	34	14	0.366	13,650	11.7	13	0.391	13,670
Sitka	40	17	0.385	8,110	9.5	10	0.406	11,150

[1] Based on weight when oven-dry and volume at test.

[2] Exact species not known.

The shape of the specimens tested in tension parallel to the grain is shown in Fig. 10. The rate of motion of the movable head of the testing machine is 0.05 in. per minute. The ultimate tensile strength is the maximum load divided by the least cross-sectional area.

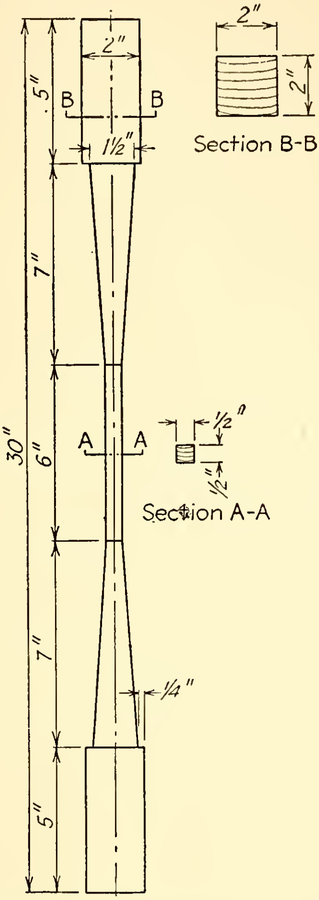

Fig. 10. Details of tension parallel to grain test specimen.

FACTORS AFFECTING THE STRENGTH OF WOOD

23. Specific Gravity. Wood is composed of elongated cells cemented together by lignin. Since the specific gravity of the material composing the cell walls, or the wood substance, is about 1.54 regardless of the species, all species would be of the same specific gravity were it not for the variation in the size and arrangement of the cell cavities. The density of the wood substance is therefore a definite index of strength properties. In Table 4 the relationship between specific gravity and strength properties is expressed as an equation based on average results of strength tests of different species. G represents the specific gravity of oven-dry wood based on the volume at the moisture content indicated.

Table 4. Relationship between Specific Gravity and Strength[1]

Property	Unit	Moisture Condition	
		Green	Air-Dry (12 Per cent Moisture Content)
Static bending:			
Fiber stress at proportional limit	Pounds per square inch	$10{,}200G^{1.25}$	$16{,}700G^{1.25}$
Modulus of rupture	Pounds per square inch	$17{,}600G^{1.25}$	$25{,}700G^{1.25}$
Work to maximum load	Inch-pounds per cubic inch	$35.6G^{1.75}$	$32.4G^{1.75}$
Total work	Inch-pounds per cubic inch	$103G^{2}$	$72.7G^{2}$
Modulus of elasticity	1,000 lb. per square inch	$2{,}360G$	$2{,}800G$
Impact bending:			
Fiber stress at proportional limit	Pounds per square inch	$23{,}700G^{1.25}$	$31{,}200G^{1.25}$
Modulus of elasticity	1,000 lb. per square inch	$2{,}940G$	$3{,}380G$
Height of drop	Inches	$114G^{1.75}$	$94.6G^{1.75}$
Compression parallel to grain:			
Fiber stress at proportional limit	Pounds per square inch	$5{,}250G$	$8{,}750G$
Maximum crushing strength	Pounds per square inch	$6{,}730G$	$12{,}200G$
Modulus of elasticity	1,000 lb. per square inch	$2{,}910G$	$3{,}380G$
Compression perpendicular to grain:			
Fiber stress at proportional limit	Pounds per square inch	$3{,}000G^{2.25}$	$4{,}630G^{2.25}$
Hardness:			
End	Pounds	$3{,}740G^{2.25}$	$4{,}800G^{2.25}$
Radial	Pounds	$3{,}380G^{2.25}$	$3{,}720G^{2.25}$
Tangential	Pounds	$3{,}460G^{2.25}$	$3{,}820G^{2.25}$

[1] The values listed in this table are to be read as equations; for example, modulus of rupture for green material = $17{,}600G^{1.25}$, where G represents the specific gravity, oven-dry, based on volume at moisture condition indicated.

The strength of pieces within the same species varies by somewhat higher powers of the specific gravities than those shown in Table 4.

In southern pine and Douglas fir the pronounced summerwood furnishes a practical means of estimating density.

24. Moisture Content. When the cell walls begin to lose moisture, or just after the wood is dried below the fiber saturation point, most strength properties increase rapidly with a decrease in moisture content. This increase is due to an actual stiffening of the cell walls as they dry out and to the compacting of the wood substance as it shrinks. All strength properties are not equally affected by changes in moisture content. The average variation of the strength properties of wood for an increase or decrease in moisture content of 1 per cent are given in Table 5.

Table 5. Average Change in Strength Properties for an Increase or Decrease in Moisture Content of 1 Per Cent

Property	*Percentage*
Static bending:	
Fiber stress at proportional limit	5
Modulus of rupture or cross-breaking strength	4
Modulus of elasticity or stiffness	2
Work to proportional limit	8
Work to maximum load or shock-resisting ability	$\frac{1}{2}$
Impact bending:	
Fiber stress at proportional limit	3
Work to proportional limit	4
Height of drop of hammer causing complete failure	$-\frac{1}{2}$
Compression parallel to grain:	
Fiber stress at proportional limit	5
Maximum crushing strength	6
Compression perpendicular to grain:	
Fiber stress at proportional limit	$5\frac{1}{2}$
Hardness, end grain	4
Hardness, side grain	$2\frac{1}{2}$
Shearing strength parallel to grain	3
Tension perpendicular to grain	$1\frac{1}{2}$

25. Rate of Growth. The width of the annual growth rings is an indication of the rate of growth but is not as efficient a criterion of strength as density or specific gravity. In certain species of wood such as Douglas fir and southern pine a definite range in the number of rings per inch is frequently associated with density. Usually very wide or very narrow growth rings contain a smaller proportion of summerwood than those of intermediate width. This means that pieces representing either extreme are less dense than average growth.

26. Position of Growth Rings. Because of the shape, size, and arrangement of the wood fibers and the amount of spring- and summerwood, the strength properties of wood vary considerably along and across the grain. For this reason wood does not follow the general behavior of an isotropic material, and its elastic properties are dependent upon the direction of the applied force with respect to the grain and the annual growth rings. From Table 2 it is apparent that the strength properties for any species are different along and across the grain. Additional tests by the Forest Products Laboratory have shown the influ-

Table 6. Average Results of Tests Showing the Influence of the Position of the Growth Rings on the Mechanical Properties of Sitka Spruce, Douglas Fir, and Loblolly Pine

Properties	Unit	Sitka Spruce				Douglas Fir				Loblolly Pine, Green	
		Green		Air-Dried		Green		Kiln-Dried			
		Position *A*[1]	Position *B*[1]	Position *A*	Position *B*	Position *A*	Position *B*	Position *A*	Position *B*	Position *A*	Position *B*
Static bending:											
Moisture	Per cent[2]	45.2	45.3	12.2	12.2	30.6	29.4	11.9	11.9	26.0	25.8
Specific gravity[3]		0.341	0.343	0.370	0.372	0.427	0.431	0.455	0.459	0.599	0.599
Fiber stress at proportional limit	Pounds per square inch	3,160	3,150	5,800	5,900	4,510	4,700	7,800	8,120	4,820	4,540
Fiber stress at maximum load	Pounds per square inch	4,890	4,960	8,470	8,450	7,280	7,470	10,630	10,860	9,750	9,740
Modulus of elasticity	1,000 lb. per sq. in.	1,104	1,124	1,370	1,374	1,475	1,480	1,723	1,713	1,398	1,398
Work to proportional limit	Inch-pounds per cubic inch	0.52	0.52	1.46	1.49	0.81	0.86	2.03	2.22	1.00	0.90
Work to maximum load	Inch-pounds per cubic inch	5.2	5.6	7.5	7.5	6.3	7.6	7.4	7.5	....	....
Work, total	Inch-pounds per cubic inch	15.8	14.4	.....	....	15.0	11.9	.....	.....	....	....
Impact bending, 50-lb. hammer:											
Moisture	Per cent[2]	45.8	44.4	12.7	12.5	30.4	30.3	10.7	11.0	....	....
Specific gravity[3]		0.343	0.350	0.372	0.378	0.422	0.431	0.457	0.460	....	....
Fiber stress at proportional limit	Pounds per square inch	7,870	7,860	10,150	9,900	8,870	9,450	12,550	12,370	....	....
Modulus of elasticity	1,000 lb. per sq. in.	1,277	1,274	1,618	1,662	1,480	1,729	2,140	2,110	....	....
Work to proportional limit	Inch-pounds per cubic inch	2.7	2.7	3.7	3.4	3.0	2.9	4.2	4.1	....	....
Height of drop causing complete failure	Inches	20	20	21.0	20.9	20.4	18.8	23.4[4]	23.1[4]	....	....
Compression parallel to grain:											
Moisture	Per cent[2]	45.2	46.4	12.7	12.7	29.6	29.2	10.2	10.4	....	....
Specific gravity[3]		0.339	0.341	0.363	0.370	0.428	0.419	0.451	0.459	....	....
Rings per inch		14.5	15.7	6.5	7.6	16.3	15.9	20.4	23.0	7.0	7.5
Fiber stress at maximum load	Pounds per square inch	2,220	2,210	4,490	4,670	3,810	3,730	7,230	7,250	4,680	4,650
Hardness[5]:											
End	Pounds	357	357	682	701	440	440	713	713	....	....
Side	Pounds	289	283	436	462	452	446	700	657	....	....
Compression perpendicular to grain:											
Fiber stress at proportional limit	Pounds per square inch	227	227	548	582	455	496	609	667	....	....
Shear parallel to grain: Shearing strength	Pounds per square inch	713	668	184	1,202	883	833	1,209	1,266	....	....
Cleavage: Cleavage strength	Pounds per square inch	122	94	242	189	133	136	163	163	....	....
Tension at right angles to grain: Tensile strength	Pounds per square inch	208	130	466	357	179	165	255	307	....	....

---

[1] Positions *A* and *B* refer to placement of growth rings with respect to directions of application of load, as illustrated in Fig. 11. [2] Percentage of moisture based on weight of oven-dry wood. [3] Specific gravity based on weight when oven-dry and volume at test. [4] Adjusted to drop for 2 by 2 in. cross section. [5] Load required to imbed a 0.444-in. ball to one-half its diameter.

ence of the position of the growth rings on the mechanical properties of Sitka spruce, Douglas fir, and loblolly pine. These are presented in Table 6, and the positions under test are illustrated in Fig. 11.

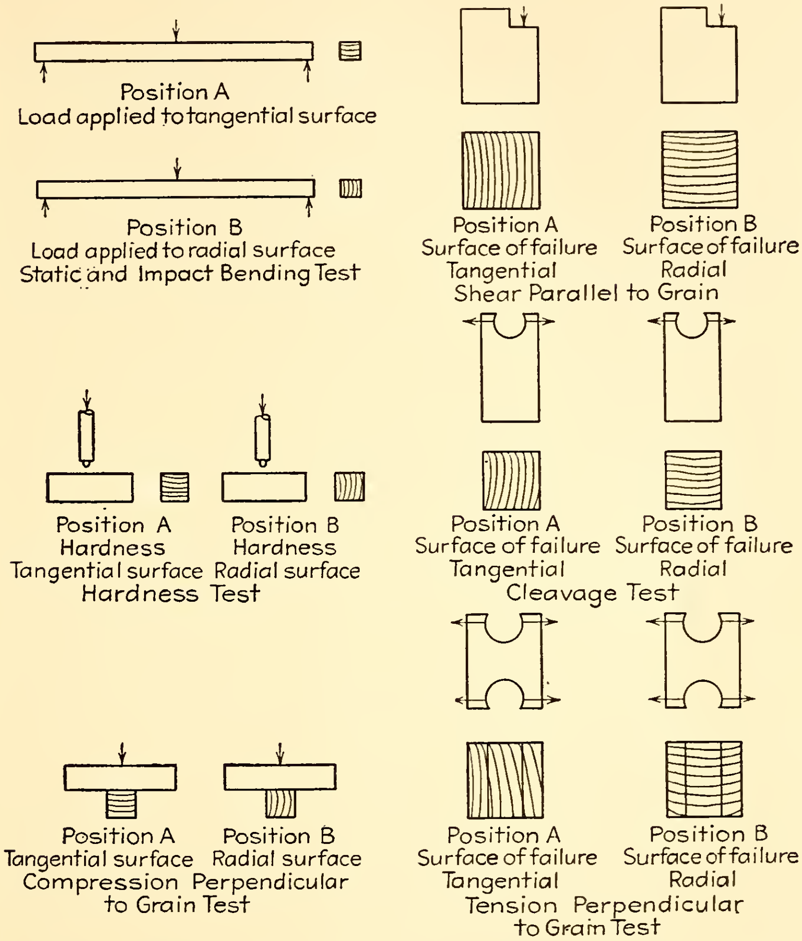
Position A
Load applied to tangential surface
Position B
Load applied to radial surface
Static and Impact Bending Test
Position A
Surface of failure
Tangential
Position B
Surface of failure
Radial
Shear Parallel to Grain
Position A
Hardness
Tangential surface
Position B
Hardness
Radial surface
Hardness Test
Position A
Surface of failure
Tangential
Position B
Surface of failure
Radial
Cleavage Test
Position A
Tangential surface
Position B
Radial surface
Compression Perpendicular
to Grain Test
Position A
Surface of failure
Tangential
Position B
Surface of failure
Radial
Tension Perpendicular
to Grain Test

FIG. 11. Method of applying load with respect to position of growth rings.

27. Knots. Knots affect strength because the grain of the wood is distorted in passing around a knot, and the direction of the fibers in the knot is approximately at right angles to those in the adjacent wood. The effect of a knot on the strength properties of a piece depends primarily on the size, shape, soundness, and location of a knot and the type of stress to which the piece is subjected.

The decrease in strength due to the presence of knots is greater when the piece is in tension rather than compression, and the influence on bending strength is dependent upon the location and the area that it occupies in the cross section of the piece. In a simply supported beam a knot on the lower side, in tension, will have a great effect on the maximum load that can be carried. Knots on the compression side are less serious, and at the neutral axis or point of maximum shear they have very little effect. Knots have practically no effect on stiffness.

In short or intermediate columns knots reduce the strength in proportion to their size, but in long columns where stiffness is the controlling factor they are of little importance.

28. Checks and Shakes. In beams the principal effect of shakes and checks is to reduce the resistance to horizontal shear. The effect of checks in beams and columns depends on the area of the longitudinal section they cover. Checks also seriously affect the resistance of a member subjected to tension perpendicular to the grain but are less injurious in straight-grained members loaded parallel to the grain.

29. Slope of Grain. When the fibers of a wood member are not parallel to the edge of the piece, cross grain exists, and the longitudinal tensile and compressive stresses have components acting across the grain. Wood is least strong across the grain, and therefore the slope of cross grain is important. The slope of grain is expressed as a ratio between a 1-in. deviation of the grain from the side of the piece and the distance within which this deviation occurs. Tests by the Forest Products Laboratory show that compressive strength is little affected until slopes of grain 1 in 10 or greater are reached. Modulus of elasticity is more affected and begins to decrease rapidly at a slope of 1 in 15. Modulus of rupture decreases even more rapidly and becomes appreciable at a slope of 1 in 20. The most pronounced effect is on shock resistance, which decreases at a slope of 1 in 25.

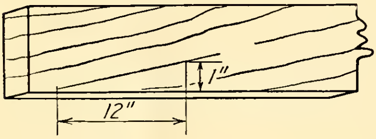

Fig. 12. Method of measuring slope of grain.

30. Live and Dead Trees. Wood cut from trees killed by insects, fungi, wind, or fire is in all respects as useful for structural lumber as wood from trees that were alive when cut. This assumes, of course, that the wood cut from the tree is sound and free from insects and decay. For this reason timber from dead trees requires careful inspection, but once it is found perfectly sound there will be no difference in the strength properties of the timber from dead trees and that from trees cut while still alive.

31. Air-Dried and Kiln-Dried Lumber. Comparative strength tests by the Forest Products Laboratory on kiln-dried and air-dried specimens show that good kiln drying and good air drying have the same effect upon the strength of wood but that severe conditions in the kiln will lower most of the strength properties. Usually the moisture content of wood on leaving the kiln is from 2 to 6 per cent lower than that of thoroughly air-dried wood. Since the strength properties of wood increase rapidly with a decrease in moisture content, kiln-dried wood might be expected to have higher strength values than air-dried wood. However, the difference in moisture content is not permanent because in actual use wood will come to the same moisture content whether it is kiln-dried or air-dried.

32. Treatments. Wood preservatives and fire-retardent treatments are not harmful to wood, but the method of injecting the chemicals, if not properly carried out, may result in a loss of strength to the wood. In any treating process care must be used with respect to the temperature and the duration of treatment. Extreme treating temperatures have a marked effect on strength properties, and the loss in strength due to high temperatures increases with moisture content and with the size of the timber undergoing treatment. Any slight decrease in strength properties that may occur because of severe treatment is offset to some extent by the extension of the life of the wood when it is used under conditions favoring decay or attack by insects or marine borers.

33. Length of Service. Too often wood is assumed to be perishable and suitable for temporary structures only. That this is not true is proved by the numerous timber structures built a century ago that are still standing in good condition. Wood, like other materials, is subject to attack by destructive agents but when properly protected from attack by fungi, insects, and marine borers, the strength properties of wood will not change with time.

34. Duration of Stress. By duration of stress is meant the time during which a load or force acts on a wooden member. This is a very important factor in the design of timber structures. In impact bending where the load is applied suddenly and then released in a few seconds, wood will resist a force more than double that required to cause failure in static bending. Conversely, wood beams under continuous loading will fail at loads one-half to three-fourths as great as those required to cause failure in a few minutes. Increases for working stresses are provided when loads remain on a structure for a short period.

Chart 3 shows the relation between the modulus of rupture and the duration of stress of Sitka spruce.

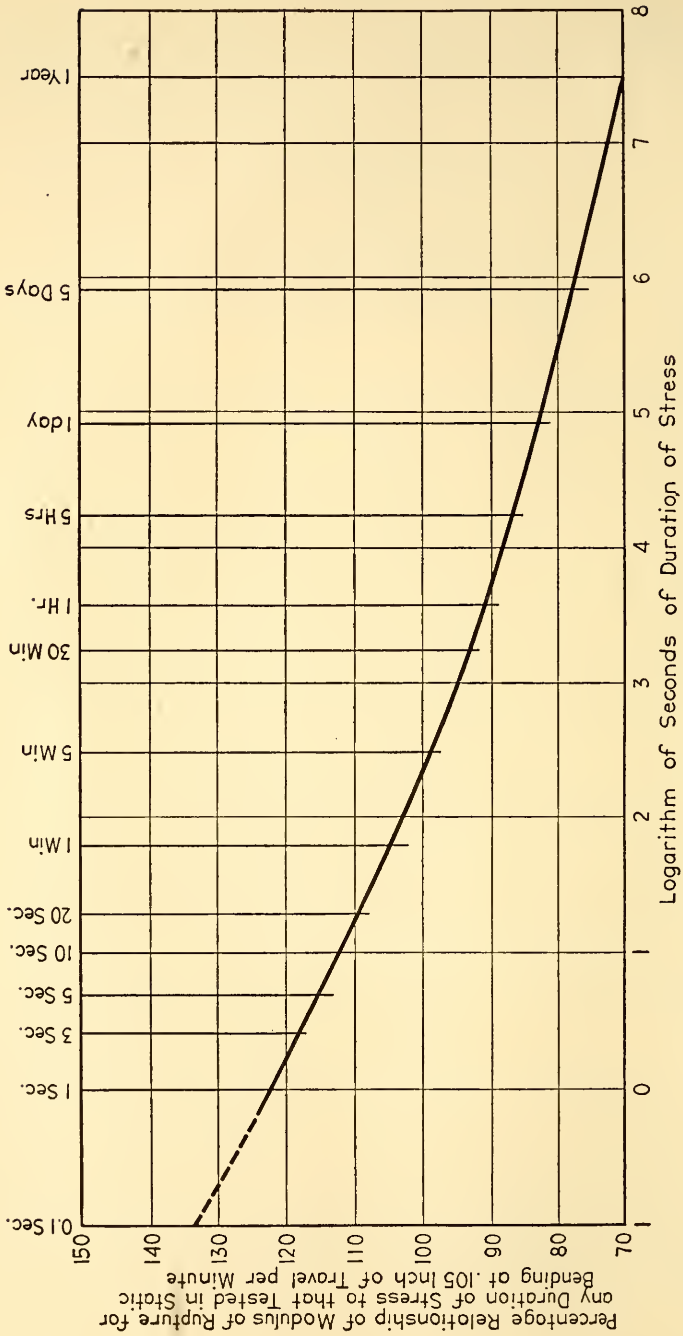

Chart 3. Variation in modulus of rupture with change in duration of stress.

CHAPTER III

GRADING RULES AND WORKING STRESSES

GRADING RULES

35. Development. The average results of tests * on approximately 250,000 small clear specimens have made it possible to assign basic stress values to commercially important woods. From these basic stresses for clear specimens and the extent to which defects and growth characteristics reduce the basic stresses, the strength of any particular piece can be determined within very close limits. Present grading rules as set forth by various lumber associations are based on United States Department of Agriculture Miscellaneous Publication 185, "Guide to the Grading of Structural Timbers and the Determination of Working Stresses," prepared at the Forest Products Laboratory.

The values given in Table 7 are the basic stresses for clear material and are *not to be used as design stresses.* These stresses are used as the basis for determining working stresses for pieces containing characteristics that influence the strength. In the derivation of basic stresses, allowances have been made for the effect of long-continued stress, the variability of the strength of clear wood of the same species, and a factor of safety.

36. Classification According to Use. Since the effect on strength of such characteristics as knots, checks, shakes, and slope of grain depends to some extent on the manner in which the piece is used, efficient grading requires that timbers be classified according to their size and use. This classification is as follows:

Beams and Stringers. Large pieces (nominal dimensions 5 by 8 in. and up) of rectangular cross section graded with respect to their strength in bending when loaded on the narrow face.

Joists and Planks. Pieces (nominal dimensions 2 to 4 in. in thickness by 4 in. and wider) of rectangular cross section graded with respect to their strength in bending when loaded either on the narrow face as joist or on the wide face as plank.

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* "Strength and Related Properties of Woods Grown in the United States," by L. J. Markwardt and T. R. C. Wilson, *Tech. Bull.* 479, U. S. Dept. Agr., 1935.

Table 7. Basic Stresses for Clear Material

(Not to be used for design purposes)

	Extreme Fiber in Bending	*Compression Perpendicular to Grain*	*Compression Parallel to Grain*	*Maximum Horizontal Shear*	*Modulus of Elasticity*
Ash, black	1,600	360	1,040	144	1,100,000
Ash, commercial white	2,240	600	1,759	202	1,500,000
Beech	2,400	600	1,920	202	1,600,000
Birch, sweet and yellow	2,400	600	1,920	202	1,600,000
Cedar, Alaska	1,760	300	1,280	144	1,200,000
Cedar, northern and southern white	1,200	210	879	112	800,000
Cedar, Port Orford	1,760	300	1,440	144	1,200,000
Cedar, western red	1,440	240	1,120	127	1,000,000
Chestnut	1,520	360	1,280	144	1,000,000
Cypress, southern	2,040	360	1,759	160	1,200,000
Douglas fir, coast region	2,400	390	1,759	144	1,600,000
Douglas fir, coast region, close-grained	2,556	414	1,880	144	1,600,000
Douglas fir, Rocky Mountain region	1,760	330	1,280	136	1,200,000
Douglas fir, dense, all regions	2,800	455	2,055	168	1,600,000
Elm, American and slippery	1,760	300	1,280	160	1,200,000
Elm, rock	2,400	600	1,920	202	1,300,000
Fir, balsam	1,440	180	1,120	112	1,000,000
Fir, commercial white	1,760	360	1,120	112	1,100,000
Gum, black and red	1,760	360	1,280	160	1,200,000
Hemlock, eastern	1,760	360	1,120	112	1,100,000
Hemlock, western	2,080	360	1,440	120	1,400,000
Hickory, true and pecan	3,040	720	2,400	224	1,800,000
Larch, western	1,920	390	1,759	160	1,300,000
Maple, sugar and black	2,400	600	1,920	202	1,600,000
Oak, commercial red and white	2,240	600	1,600	202	1,500,000
Pine, western white, northern white, ponderosa, and sugar	1,440	300	1,200	136	1,000,000
Pine, Norway	1,760	360	1,280	136	1,200,000
Pine, southern yellow	2,400	390	1,759	175	1,600,000
Pine, southern yellow, dense	2,798	455	2,063	205	1,600,000
Redwood	1,920	300	1,600	112	1,200,000
Redwood, close-grained	2,046	320	1,707	112	1,200,000
Spruce, Engelmann	1,200	210	960	112	800,000
Spruce, red, white, and Sitka	1,760	300	1,280	136	1,200,000
Tamarack	1,920	360	1,600	151	1,300,000
Tupelo	1,760	360	1,280	160	1,200,000

Posts and Timbers. Pieces of square or approximately square cross section, 4 by 4 in. and larger, in nominal dimension, graded primarily for use as posts or columns but adapted to miscellaneous uses in which strength in bending is not especially important.

The principal factors which must be limited in structural grades are knots, checks and shakes, and slope of grain.

37. Knots in Beams and Stringers. The influence of a knot in a beam is determined by its location and the area of its projection on the cross section of the piece. On the narrow face of beams and stringers the size of a knot is taken as its width between lines enclosing the knot and parallel to the edges of the piece, except that, when a knot on a narrow face extends into the adjacent one-fourth of the width of a wide face, its least dimension is taken as its size.

On a wide face the size of a knot is taken as its smallest diameter.

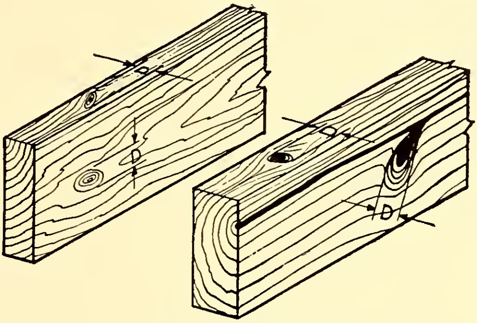

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FIG. 13. Measurement of knots in beams and stringers.

At the edges of wide faces knots are limited to the same sizes as on the narrow faces of the same piece.

The sizes of knots on narrow faces and at edges of wide faces increase proportionately from the size permitted in the middle third of the length to twice that size at the ends of the piece, except that the size of no knot should exceed the size permitted along the centerline of the wide face.

The size of knots on wide faces increases proportionately from the size permitted at the edge to the size permitted along the centerline.

The sum of the sizes of all knots within the middle half of the length of any face measured as specified for the face under consideration should not exceed four times the size of the largest knot allowed on that face.

Cluster knots and knots in groups are not permitted. Knot holes and holes from other causes are limited and measured as knots are.

38. Knots in Joists and Planks. On the narrow face of joists and planks, the size of a knot is taken as its width between lines parallel to the edges of the piece. The only knots measured on the narrow faces of a piece are those that do not show on the wide faces.

On the wide face the size of knot is taken as half the sum of its largest and smallest diameters, and the size of a spike knot is taken as half the sum of its length and greatest width.

The sizes of knots on narrow faces and at edges of wide faces increase proportionately from the size permitted in the middle third of the length to twice that size at the ends of the piece.

The size of knots on wide faces increases proportionately from the size permitted at the edge to the size permitted along the centerline.

The sum of the sizes of all knots within the middle half of the length of any face measured as specified for the face under consideration should not exceed four and one-half times the size of the largest knots allowed on that face.

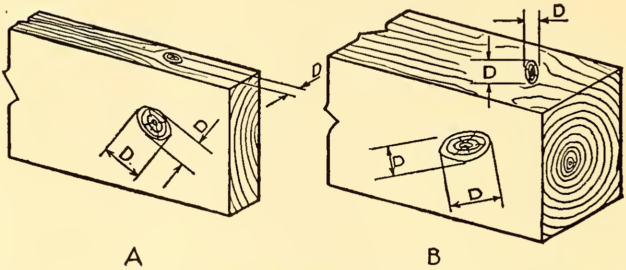

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FIG. 14. Measurement of knots (*a*) on joists and planks, and (*b*) on posts and timbers.

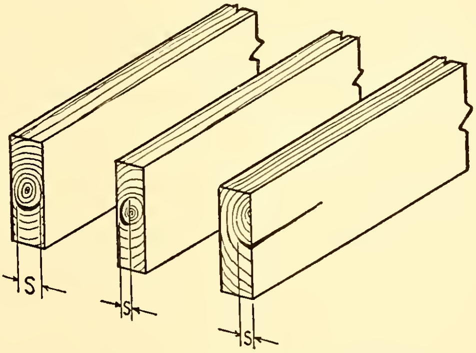
39. Knots in Posts and Timbers. On any face the size of a knot in posts and timbers is taken as one-half the sum of its largest and smallest diameter, and the size of a spike knot is taken as one-half the sum of its length and greatest width.

The sum of the sizes of all knots in any 6 in. of length of the piece is not permitted to exceed twice the maximum permissible size of knots. Two knots of maximum permissible size are not allowed in the same 6 in. on any face.

40. Shakes and Checks. In joists and planks and beams and stringers, shake in green material is permitted in direct proportion of increase from none in a grade of the strength of green, clear wood to one-half the width of the piece in a grade having one-half the strength of green clear wood. In seasoned material, shake is similarly permitted from one-ninth the width of the piece in a grade of the strength of green, clear wood to five-ninths the width of the piece in a grade having one-half the strength of green, clear wood.

Shakes and checks, as a rule, have little influence on the strength of columns unless so extensive as to split a column practically in two. Limitation on shakes and checks in columns is more important from the standpoints of appearance and of reducing opportunity for decay. In post and timber grades, shake in green material is permitted in direct proportion of increase from two-tenths the width of the piece in a grade having the strength of green, clear wood to six-tenths the width of the piece in a grade having one-half the strength of green, clear wood. In seasoned material, shake is similarly permitted from three-tenths the width of the piece in a grade having the strength of green, clear wood to seven-tenths the width of the piece in a grade having one-half the strength of green, clear wood.

Shakes are measured at the ends of the piece. In beams and stringers and joists and planks only those within the middle half of the height of the piece are considered. The size of the shake is the distance between lines enclosing the shake and parallel to the wide faces of the piece.



Courtesy Timber Engineers' Handbook,
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FIG. 15. Measurement of shakes in beams and stringers or joists and planks.

The size of checks is taken within the middle half of the height of the piece and within three times the height from the end, and is the estimated area, along the horizontal section showing maximum area, divided by three times the height of the piece. When checks on two parallel faces are opposite, the sum of their sizes is taken. The sum of the sizes of shakes and checks should not exceed the permissible size of shake.

In posts and timbers, shakes are measured at the ends of the piece, and the size is the distance between the maximum spaced pair of lines exactly enclosing the shakes and parallel to two opposite faces. The size of checks within three times the width of the piece from either end shall be taken as their estimated area, along the longitudinal section showing maximum area, divided by three times the width of the piece. The sum of the sizes of shakes and checks should not exceed the permissible size of shake.

Checks extending entirely across the end should not extend into the piece, at the center of the width of the end, a distance greater than the size of the allowable shake.

41. Slope of Grain. Slope of grain, resulting either from diagonal sawing or from spiral or twisted grain in the log, is limited in accordance with the recommendations of the Forest Products Laboratory, which are based on the results of detailed study of the effect of cross and spiral grain upon strength and the weakening of material by checks, which invariably develop and, without exception, follow the grain.

Slope of grain should be measured over a distance which will assure the determination of the general slope of the grain not influenced by short, local deviations. In a piece in bending, it is of greatest importance in the upper and lower quarters and the middle half of the length. If it meets the limitations of a grade in one location, it may be somewhat greater elsewhere.

42. Wane. Wane, which is bark or the lack of wood on the edges of a piece, is limited by such considerations as bearing area, nailing edge, and appearance, rather than by effect on strength. The percentage reduction in strength resulting from wane toward the center of a beam is about double the percentage reduction in cross-sectional area. No combination of wane and knots is permitted which would reduce the strength more than the maximum allowable knot. The occurrence of maximum wane and maximum knot in the same cross section at the center of a beam is so rare, however, and the effect of allowable maximum wane is so small a percentage of the effect of maximum allowable knot that the additional reduction in strength beyond the effect of the knot would be slight, and it is usually unnecessary to give attention to combination of wane and knots.

43. Pitch Pockets. Pitch pockets are ordinarily not defects in a structural grade. A large number, however, indicate a general lack of bond, and such a piece should be carefully examined for shakes.

44. Grading Rules. The results of tests on small specimens of clear wood and additional data on the factors affecting the strength of large structural timbers has made it possible to establish definite principles for the grading of timbers for strength. The classification of structural timbers into grades within which reasonable uniformity and minimums of strength are obtained is the basis for assigning working stresses to the various grades. The structural grades produced by associations of lumber manufacturers are listed, along with their working stresses, in the grading rules published by these associations. Each association represents one or more species, and those with published grading rules for structural grades are given below.

Association	*Species*
National Hardwood Lumber Association	Ash, white Beech Birch Chestnut Cypress, southern Elm, rock Elm, soft Gum, black and red Hickory Maple, hard Oak, red and white Pecan Poplar, yellow Tupelo
Southern Cypress Association	Cypress, tidewater red
West Coast Bureau of Lumber Grades and Inspection	Douglas fir, coast region Hemlock, West Coast
Western Pine Association	Douglas fir, inland empire Larch
Northern Hemlock and Hardwood Manufacturers Association	Hemlock, eastern Pine, Norway
Southern Pine Inspection Bureau of Southern Pine Association	Pine, southern longleaf Pine, southern shortleaf
California Redwood Association	Redwood
Northeastern Lumber Manufacturers Association	Spruce, eastern

WORKING STRESSES

45. Values. Table 8 lists the working stresses for standard commercial grades of lumber. The unit stresses should be used in design only after they have been modified in accordance with the accompanying text.

46. Conditions of Exposure. The allowable working stresses in Table 8 apply to timber under any conditions of moisture content or seasoning, and no modification of these stresses should be made when

Table 8. Working Stresses

Species and Commercial Grade[1]		Rules under Which Graded	Allowable Unit Stresses in Pounds per Square Inch				Modulus of Elasticity E
			Extreme Fiber in Bending f and Tension Parallel to Grain[2] t	Horizontal Shear H	Compression Perpendicular to Grain $c\perp$	Compression Parallel to Grain[3] c	
Ash, white:		National Hardwood Lumber Association			600		1,500,000
2150 f grade	J.&P.		2,150	145		1,700	
1900 f grade	J.&P.-B.&S.		1,900	145		1,500	
1700 f grade	J.&P.-B.&S.		1,700	145		1,325	
1450 f grade	J.&P.-B.&S.		1,450	120		1,150	
1300 f grade	B.&S.		1,300	120		1,050	
1450 c grade	P.&T.		.....	...		1,450	
1200 c grade	P.&T.		.....	...		1,200	
1075 c grade	P.&T.		.....	...		1,075	
Beech:		National Hardwood Lumber Association			600		1,600,000
2150 f grade	J.&P.		2,150	145		1,750	
1900 f grade	J.&P.-B.&S.		1,900	145		1,525	
1700 f grade	J.&P.-B.&S.		1,700	145		1,350	
1450 f grade	J.&P.-B.&S.		1,450	120		1,150	
1550 c grade	P.&T.		.....	...		1,550	
1450 c grade	P.&T.		.....	...		1,450	
1200 c grade	P.&T.		.....	...		1,200	
Birch:		National Hardwood Lumber Association			600		1,600,000
2150 f grade	J.&P.		2,150	145		1,750	
1900 f grade	J.&P.-B.&S.		1,900	145		1,525	
1700 f grade	J.&P.-B.&S.		1,700	145		1,350	
1450 f grade	J.&P.-B.&S.		1,450	120		1,150	
1550 c grade	P.&T.		.....	...		1,550	
1450 c grade	P.&T.		.....	...		1,450	
1200 c grade	P.&T.		.....	...		1,200	
Chestnut:		National Hardwood Lumber Association			360		1,000,000
1450 f grade	J.&P.		1,450	120		1,200	
1200 f grade	J.&P.-B.&S.		1,200	120		950	
1075 c grade	P.&T.		.....	...		1,075	
Cypress, southern:		National Hardwood Lumber Association			360		1,200,000
1700 f grade	J.&P.-B.&S.		1,700	145		1,425	
1300 f grade	J.&P.-B.&S.		1,300	120		1,125	
1450 c grade	P.&T.		.....	...		1,450	
1200 c grade	P.&T.		.....	...		1,200	
Cypress, tidewater red:		Southern Cypress Manufacturers Association			360		1,200,000
1700 f grade	J.&P.-B.&S.		1,700	145		1,425	
1300 f grade	J.&P.-B.&S.		1,300	120		1,125	
1450 c grade	P.&T.		.....	...		1,450	
1200 c grade	P.&T.		.....	...		1,200	

Table 8. Working Stresses—*Continued*

Species and Commercial Grade [1]		Rules under Which Graded	Allowable Unit Stresses in Pounds per Square Inch				Modulus of Elasticity E
			Extreme Fiber in Bending f and Tension Parallel to Grain [2] t	Horizontal Shear H	Compression Perpendicular to Grain $c\perp$	Compression Parallel to Grain [3] c	
Douglas fir, coast region:		West Coast Bureau of Lumber Grades and Inspection					1,600,000
2150 f dense select structural	J.&P.		2,150	145	455	1,550	
2150 f dense select structural	B.&S.		2,150	145	455	1,550	
1900 f select structural	J.&P.		1,900	120	415	1,450	
1900 f select structural	B.&S.		1,900	120	415	1,450	
1700 f dense no. 1	J.&P.		1,700	145	455	1,325	
1700 f dense no. 1	B.&S.		1,700	145	455	1,325	
1450 f no. 1	J.&P.		1,450	120	390	1,200	
1450 f no. 1	B.&S.		1,450	120	390	1,200	
1100 f no. 2	J.&P.		1,100	110	390	1,075	
1550 c dense select structural	P.&T.		.....	...	455	1,550	
1450 c select structural	P.&T.		.....	...	415	1,450	
1400 c dense no. 1	P.&T.		.....	...	455	1,400	
1200 c no. 1	P.&T.		.....	...	390	1,200	
Douglas fir, inland empire:		Western Pine Association					
Select structural	J.&P.		2,150	145	455	1,750	1,600,000
Structural	J.&P.		1,900	100	400	1,400	1,500,000
Common structural	J.&P.		1,450	95	380	1,250	1,500,000
Select structural	P.&T.		.....	...	455	1,750	1,600,000
Structural	P.&T.		.....	...	400	1,400	1,500,000
Common structural	P.&T.		.....	...	380	1,250	1,500,000
Elm, rock:		National Hardwood Lumber Association			600		1,300,000
2150 f grade	J.&P.		2,150	145		1,750	
1900 f grade	J.&P.–B.&S.		1,900	145		1,525	
1700 f grade	J.&P.–B.&S.		1,700	145		1,350	
1450 f grade	J.&P.–B.&S.		1,450	120		1,150	
1550 c grade	P.&T.		.....	...		1,550	
1450 c grade	P.&T.		.....	...		1,450	
1200 c grade	P.&T.		.....	...		1,200	
Elm, soft:		National Hardwood Lumber Association			300		1,200,000
1700 f grade	J.&P.		1,700	120		1,225	
1450 f grade	J.&P.–B.&S.		1,450	120		1,050	
1200 f grade	J.&P.–B.&S.		1,200	120		875	
1075 c grade	P.&T.		.....	...		1,075	
Gum, black and red:		National Hardwood Lumber Association			360		1,200,000
1700 f grade	J.&P.		1,700	120		1,225	
1450 f grade	J.&P.–B.&S.		1,450	120		1,050	
1200 f grade	J.&P.–B.&S.		1,200	120		875	
1075 c grade	P.&T.		.....	...		1,075	

Table 8. Working Stresses—*Continued*

Species and Commercial Grade[1]		Rules under Which Graded	Allowable Unit Stresses in Pounds per Square Inch				Modulus of Elasticity E
			Extreme Fiber in Bending f and Tension Parallel to Grain[2] t	Horizontal Shear H	Compression Perpendicular to Grain $c\perp$	Compression Parallel to Grain[3] c	
Hemlock, eastern:		Northern Hemlock and Hardwood Manufacturers Association			360		1,100,000
Select structural	J.&P.-B.&S.		1,300	85		850	
Prime structural	J.&P.		1,200	60		775	
Common structural	J.&P.		1,100	60		650	
Utility structural	J.&P.		950	60		600	
Select structural	P.&T.		.....	...		850	
Hemlock, west coast:		West Coast Bureau of Lumber Grades and Inspection			360		1,400,000
1600 f select structural	J.&P.		1,600	100		1,100	
1450 f no. 1	B.&S.		1,450	100		1,075	
1450 f no. 1	J.&P.		1,450	100		1,075	
1100 f no. 2	J.&P.		1,100	90		850	
1075 c no. 1	P.&T.		.....	...		1,075	
Hickory:		National Hardwood Lumber Association			720		1,800,000
2150 f grade	J.&P.-B.&S.		2,150	145		1,725	
1900 f grade	J.&P.-B.&S.		1,900	145		1,550	
1700 f grade	J.&P.-B.&S.		1,700	145		1,350	
1550 c grade	P.&T.		.....	...		1,550	
1450 c grade	P.&T.		.....	...		1,450	
1325 c grade	P.&T.		.....	...		1,325	
Larch:		Western Pine Association					1,300,000
Select structural	J.&P.		2,150	145	455	1,750	
Structural	J.&P.		1,900	120	415	1,450	
Common structural	J.&P.		1,450	120	390	1,325	
Select structural	P.&T.		.....	...	455	1,750	
Structural	P.&T.		.....	...	415	1,450	
Common structural	P.&T.		.....	...	390	1,325	
Maple, hard:		National Hardwood Lumber Association			600		1,600,000
2150 f grade	J.&P.		2,150	145		1,750	
1900 f grade	J.&P.-B.&S.		1,900	145		1,525	
1700 f grade	J.&P.-B.&S.		1,700	145		1,425	
1450 f grade	J.&P.-B.&S.		1,450	120		1,150	
1550 c grade	P.&T.		.....	...		1,550	
1450 c grade	P.&T.		.....	...		1,450	
1200 c grade	P.&T.		.....	...		1,200	
Oak, red and white:		National Hardwood Lumber Association			600		1,500,000
2150 f grade	J.&P.		2,150	145		1,550	
1900 f grade	J.&P.-B.&S.		1,900	145		1,375	
1700 f grade	J.&P.-B.&S.		1,700	145		1,200	
1450 f grade	J.&P.-B.&S.		1,450	120		1,050	
1300 f grade	B.&S.		1,300	120		950	
1325 c grade	P.&T.		.....	...		1,325	
1200 c grade	P.&T.		.....	...		1,200	
1075 c grade	P.&T.		.....	...		1,075	

Table 8. Working Stresses—*Continued*

Species and Commercial Grade[1]		Rules under Which Graded	Allowable Unit Stresses in Pounds per Square Inch				Modulus of Elasticity E
			Extreme Fiber in Bending f and Tension Parallel to Grain[2] t	Horizontal Shear H	Compression Perpendicular to Grain $c\perp$	Compression Parallel to Grain[3] c	
Pecan:		National Hardwood Lumber Association			720		1,800,000
2150 f grade	J.&P.-B.&S.		2,150	145		1,725	
1900 f grade	J.&P.-B.&S.		1,900	145		1,550	
1700 f grade	J.&P.-B.&S.		1,700	145		1,350	
1550 c grade	P.&T.		.....	...		1,550	
1450 c grade	P.&T.		.....	...		1,450	
1325 c grade	P.&T.		.....	...		1,325	
Pine, Norway:		Northern Hemlock and Hardwood Manufacturers Association			360		1,200,000
Prime structural	J.&P.		1,200	75		900	
Common structural	J.&P.		1,100	75		775	
Utility structural	J.&P.		950	75		650	
Pine, southern longleaf:		Southern Pine Inspection Bureau of the Southern Pine Association			455		1,600,000
Select structural	J.&P.-B.&S.		2,400	120		1,750	
Prime structural	J.&P.-B.&S.		2,000	120		1,400	
Merchantable structural	J.&P.-B.&S.		1,800	120		1,300	
Structural S.E.&S.	J.&P.-B.&S.		1,800	120		1,300	
No. 1 structural	J.&P.-B.&S.		1,600	120		1,150	
No. 1 dimension	J.&P.		1,700	150		1,400	
No. 2 stress dimension	J.&P.		1,250	100		1,025	
Select structural	P.&T.		.....	...		1,750	
Prime structural	P.&T.		.....	...		1,400	
Merchantable structural	P.&T.		.....	...		1,300	
Structural S.E.&S.	P.&T.		.....	...		1,300	
No. 1 structural	P.&T.		.....	...		1,150	
Pine, southern shortleaf:		Southern Pine Inspection Bureau of the Southern Pine Association					1,600,000
Dense select structural	J.&P.-B.&S.		2,400	120	455	1,750	
Dense structural	J.&P.-B.&S.		2,000	120	455	1,400	
Dense structural S.E.&S.	J.&P.-B.&S.		1,800	120	455	1,300	
Dense No. 1 structural	J.&P.-B.&S.		1,600	120	455	1,150	
No. 1 dense dimension	J.&P.		1,700	150	455	1,400	
No. 1 dimension	J.&P.		1,450	125	390	1,200	
No. 2 dense stress dimension	J.&P.		1,250	100	455	1,025	
No. 2 medium grain stress dimension	J.&P.		1,100	85	390	875	
Dense select structural	P.&T.		.....	...	455	1,750	
Dense structural	P.&T.		.....	...	455	1,400	
Dense structural S.E.&S.	P.&T.		.....	...	455	1,300	
Dense No. 1 structural	P.&T.		.....	...	455	1,150	

Table 8. Working Stresses—*Continued*

Species and Commercial Grade[1]		Rules under Which Graded	Allowable Unit Stresses in Pounds per Square Inch				Modulus of Elasticity E
			Extreme Fiber in Bending f and Tension Parallel to Grain[2]	Horizontal Shear H	Compression Perpendicular to Grain $c\perp$	Compression Parallel to Grain[3] c	
Poplar, yellow:		National Hardwood Lumber Association			300		1,100,000
1500 f grade	J.&P.		1,500	110		1,200	
1250 f grade	J.&P.-B.&S.		1,250	110		950	
1075 c grade	P.&T.		.....	...		1,075	
Redwood:		California Redwood Association					1,200,000
Dense structural	J.&P.-B.&S.		1,700	110	320	1,450	
Heart structural	J.&P.-B.&S.		1,300	95	320	1,100	
Spruce, eastern:		Northeastern Lumber Manufacturers Association, Inc.			300		1,200,000
1450 f structural grade	J.&P.		1,450	110		1,050	
1300 f structural grade	J.&P.		1,300	95		975	
1200 f structural grade	J.&P.		1,200	95		900	
Tupelo:		National Hardwood Lumber Association			360		1,200,000
1700 f grade	J.&P.		1,700	120		1,225	
1450 f grade	J.&P.-B.&S.		1,450	120		1,050	
1200 f grade	J.&P.-B.&S.		1,200	120		875	
1075 c grade	P.&T.		.....	...		1,075	

[1] Abbreviations: J.&P., Joists and Planks; B.&S., Beams and Stringers; P.&T., Posts and Timbers; S.E.&S., Square Edge and Sound.

[2] When graded in accordance with par. 52.

[3] When graded according to par. 50 and when the l/d ratio is 11 or less.

the timber is used in locations favoring decay. The designing engineer should take care of decay hazards by proper design, maintenance, selection or treatment of lumber, or other methods.

47. Duration of Load. Where a member is fully stressed to the maximum allowable stress for many years, either continuously or cumulatively under the condition of maximum design load, use working stresses 90 per cent of those in Table 8. When the duration of the full maximum load does not exceed the period indicated, increase the allowable unit stresses in Table 8 except modulus of elasticity, as follows:

15 per cent for 2 months duration, as for snow.
25 per cent for 7 days duration.

33⅓ per cent for wind or earthquake.
100 per cent for impact.

48. Impact. The working stresses, except modulus of elasticity, may be used without regard to impact if the stress induced by impact does not exceed the allowable working stress for permanent loading. However, the sum of stresses induced by any combination of impact and other loading should not exceed twice the allowable working stress.

49. Flexure. For joist and plank grades the working stresses in flexure apply to material with the load applied to either the narrow or wide face; for beam and stringer grades they apply only to material loaded on the narrow face.

The grading rules for beams apply to simple beams and need modification for continuous beams. When a beam has more than one span, the grading provisions applicable to the middle third of the length of simple beams should be applied to the middle two-thirds of the length of beams with two spans and to the entire length of beams of three or more spans.

50. Compression Parallel to the Grain. Values in compression parallel to the grain for post and timber grades apply to columns having an l/d ratio of 11 or less. They also apply to joist and plank and beam and stringer grades when used as columns with an l/d ratio of 11 or less, provided that the following additional provisions are added: The sum of the sizes of all knots in any 6 in. of the length of the piece should not exceed twice the maximum permissible size of knot. Two knots of maximum permissible size should not be within the same 6 in. of length on any face.

Where the l/d ratio is greater than 11, suitable column formulas must be used.

51. Tension. For members in direct tension only, the allowable stresses for the respective grades are the same as for extreme fiber stress in bending.

52. Combined Stresses. Joist and plank or beam and stringer grades should be used for members subjected to combined flexure and axial compressive stresses. When members are subjected to combined flexure and tension, the allowable working stress in tension for joist and plank and beam and stringer grades should be used, provided that the following additional provisions are added: The sum of the sizes of all knots in any 6 in. of the length of the piece should not exceed twice the maximum permissible size of knot. Two knots of maximum permissible size shall not be within the same 6 in. of length of any face.

53. Shear. The allowable working stresses for shear in joint details should be 50 per cent greater than the horizontal shear values otherwise permitted.

54. Compression Perpendicular to the Grain. If joists are supported by a ribbon or ledger board and spiked to the studs, the values for compression perpendicular to grain should be increased by 50 per cent.

The values for compression perpendicular to the grain apply to bearings 6 in. or more in length, located anywhere in the length of the piece. For bearings less than 6 in. in length and located 3 in. or more from the end of a timber, the working stress may be increased in accordance with Table 9.

Table 9. Increase in Working Stress for Different Lengths of Bearings

Length of bearing (inches)	$\frac{1}{2}$	1	$1\frac{1}{2}$	2	3	4	6 or more
Factor	1.75	1.38	1.25	1.19	1.13	1.10	1.00

55. Compression at an Angle to the Grain. When the direction of pressure is at an angle to the grain the allowable unit stress must be calculated by the Hankinson formula,

$$N = \frac{pq}{p \sin^2 \theta + q \cos^2 \theta}$$

where N = allowable unit stress on the inclined surface.
p = unit stress in compression parallel to grain.
q = unit stress in compression perpendicular to grain.
θ = angle between the direction of the load and the direction of the grain.

56. Bearing on Metal Plates. If a compression member bears on a metal plate or strap, such as at the heel joint of a truss, and contains no knot at the bearing greater in diameter than 1½ in. or one-fourth the minimum cross-sectional dimension of the member, whichever is less, the allowable unit stress for bearing will be the value given in Table 10.

57. End-to-End Bearing. For compression members bearing end-to-end, the full allowable compressive stress parallel to the grain may be used provided that there is adequate lateral support, that the end cuts of the members are square and parallel, and that a metal plate of not less than 20-gage is inserted with a drive fit between the abutting ends.

Table 10. Allowable Unit Stress in Bearing Parallel to Grain on Metal Plates, in Pounds per Square Inch

Species	
Ash, white	1625
Beech	1750
Birch	1750
Chestnut	1175
Cypress, southern	1625
Cypress, tidewater	1625
Douglas fir, all regions, dense	1875
Douglas fir, coast region, close-grained	1725
Douglas fir, coast region	1625
Douglas fir, inland region, close-grained	1600
Douglas fir, inland region	1500
Elm, rock	1750
Elm, soft	1175
Gum, black and red	1175
Hemlock, eastern	1025
Hemlock, western	1325
Hickory	2200
Larch, dense	1875
Larch, close-grained	1725
Larch	1625
Maple, hard	1750
Oak, red and white	1475
Pecan	2200
Pine, Norway	1175
Pine, southern longleaf	1875
Pine, southern shortleaf, dense	1875
Pine, southern shortleaf	1625
Poplar, yellow	1175
Redwood, close-grained	1575
Redwood	1475
Spruce, eastern	1175
Tupelo	1175

CHAPTER IV

MECHANICS OF WOOD

58. General Considerations. In the analysis of stresses and strains set up by external forces acting on a body of steel, the material is assumed to be isotropic. This means that steel has the same elastic properties in all directions; and, although it may not be considered isotropic in the strictest sense, it does behave under a load in such a manner as to follow the laws and formulas derived for such a material. Wood, however, is not isotropic. The results of tests given in Table 2 show that wood does not follow the general behavior of an isotropic material and that its elastic properties are dependent upon the direction of the applied force with respect to the grain and the annual growth rings.

The location and character of the fibers make wood an anisotropic material, but for all practical purposes wood may be considered as orthotropic. As such, it is a material having three mutually perpendicular planes of elastic symmetry.

59. Planes of Elastic Symmetry. The visible structure of wood shows the grain running in one direction and the growth rings in a plane perpendicular to the grain. Since the strength properties of wood depend upon the direction of the applied load with respect to the grain direction and the position of the annual growth rings, it is apparent that wood has three perpendicular planes of elastic symmetry: longitudinal, tangential, and radial. The longitudinal axis is parallel to the grain, the tangential axis is tangent to the growth rings, and the radial axis is perpendicular to the growth rings. These are illustrated in Fig. 16.

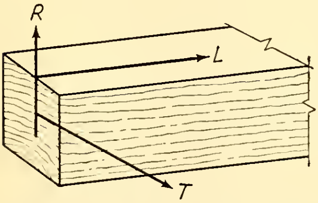

Fig. 16. Three principal axes of wood.

60. Notations. The forces acting on a piece of wood may be resolved into the three components parallel to the natural axes L, T, and R. The letter f will denote normal stress and the letter S shear stress. The letter e will be used to denote strain, and Poisson's ratio will be represented by the letter V. The subscripts L, T, and R will be used to indicate the direction of stress and strain. The modulus of elasticity will be signified by the letter E, and for the modulus of rigidity in shear the letter G will be used.

Normal stresses will be considered positive when producing tension and negative when producing compression. The normal and shearing stresses may be represented by a small cubic element like that shown in Fig. 17. The stresses on the opposite faces of the cube are represented as dotted lines. They are, of course, equal in magnitude, but opposite in sign, to the stresses shown.

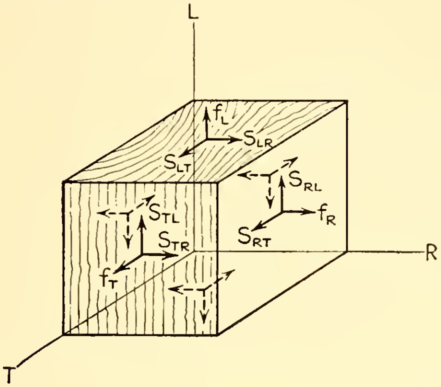

Fig. 17. Notations for the components of stress acting on a cubic block of wood.

61. Relation between Stress and Strain. The stresses acting on the six sides of the cubic element in Fig. 17 are denoted by three normal stresses, f_L, f_T, and f_R and six shearing stresses, S_{LT}, S_{LR}, S_{RT}, S_{RL}, S_{TL}, and S_{TR}. In order to keep the element in equilibrium it can be shown that these six shearing stresses reduce to three so that

$$S_{TR} = S_{RT} \qquad S_{LT} = S_{TL} \qquad S_{LR} = S_{RL}$$

The combinations of the three shearing stresses and the three normal stresses are sufficient to describe the stresses acting on the orthotropic planes through a point in any wood member. If, then, the orthotropic axes are chosen as axes of reference and the cubic block of wood is acted upon by the three normal stresses f_L, f_T, and f_R, uniformly distributed over the sides, the resultant components of strain may be found by superposing the strain components produced by each of the three stresses. If only one stress, f_L, is acting, the strains will be as follows:

$$e_L = \frac{f_L}{E_L} \qquad e_T = V_{LT}\frac{f_L}{E_L} \qquad e_R = V_{LR}\frac{f_L}{E_L}$$

where V_{LT} = Poisson's ratio, contraction along T to extension along L due to tension acting along L.

V_{LR} = Poisson's ratio, contraction along R to extension along L due to tension acting along L.

By the method of superposition, the following values for strains result when all three stresses are acting.

$$e_L = \frac{f_L}{E_L} - V_{TL}\frac{f_T}{E_T} - V_{RL}\frac{f_R}{E_R}$$

$$e_T = \frac{f_T}{E_T} - V_{RT}\frac{f_R}{E_R} - V_{LT}\frac{f_L}{E_L}$$

$$e_R = \frac{f_R}{E_R} - V_{LR}\frac{f_L}{E_L} - V_{TR}\frac{f_T}{E_T}$$

If shearing stresses are acting on the sides of the block of wood shown in Fig. 17, the distortion of the angle between any two axes depends only on the shearing stress components parallel to these axes, and the values of the shearing strains are as follows:

$$e_{LT} = \frac{S_{TL}}{G_{LT}} \qquad e_{RL} = \frac{S_{LR}}{G_{RL}} \qquad e_{TR} = \frac{S_{RT}}{G_{TR}}$$

From the equations given for strains, along the L, T, and R axes and the shearing strains, it is evident that twelve elastic constants must be known to describe completely an orthotropic material. These are three moduli of elasticity, three moduli of rigidity in shear, and six Poisson's ratios. Certain relationships exist, however, between these constants which reduce the required number to nine. These relationships are as follows:

$$\frac{V_{TL}}{E_T} = \frac{V_{LT}}{E_L} \qquad \frac{V_{RL}}{E_R} = \frac{V_{LR}}{E_L} \qquad \frac{V_{RT}}{E_R} = \frac{V_{TR}}{E_T}$$

The complete elastic constants have been determined for only a few species of wood. These are given in Table 11.

The foregoing relations between stress and strain are based on the stresses acting along the natural axes of the wood; that is, the longitudinal axis L parallel to the grain, the tangential axis T tangent to the growth rings, and the radial axis R perpendicular to the growth rings. When the stresses are inclined to these axes, the mathematical solution *

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* "Stress-Strain Relations in Wood and Plywood Considered as Orthotropic Materials," by H. W. March, *Mimeo.* 1503, U. S. Dept. Agr., 1944.

Table 11. Elastic Constants for Wood

["Report on Materials of Construction Used in Aircraft," by C. F. Jenkins, Aeronautical Research Committee (British), 1920]

	Compression				Tension			Shear			Poisson's Ratio			Constants			Shear Moduli $\times 10^4$ (from Shear Tests)			Shear Moduli $\times 10^4$ (Calculated from 45° Compression Test)		
	—	*L*	*R*	*T*	*L*	*R*	*T*	*LT*	*LR*	*RT*							*LT*	*LR*	*RT*	*LT*	*LR*	*RT*
Spruce:	$E \times 10^6$	1.95	0.13	0.07	.....	....	....	....	....	......	V_{LR} 0.45 V_{RL} 0.03	V_{LT} 0.539 V_{TL} 0.0194	V_{RT} 0.559 V_{TR} 0.301	*a* 0.156 *d* 0.0648	*b* 0.0897 *e* 0.09	*c* 2.027 *f* 0.0506						
Density 27 lb. per cu. ft.	El. Lt.[1]	5,900	320	290	.....	....	....	....	....	......							7.2	10.4	0.46	9.3	15.3	0.53
Moisture 12.2%	Ult.	7,000	700	900	18,000	800	600	1,100	1,200	300 approx.												
Light Mahogany:	$E \times 10^6$	1.8	0.14	0.07	.....	....	....	....	....	......	V_{LR} 0.31 V_{RL} 0.0241	V_{LT} 0.552 V_{TL} 0.0215	V_{RT} 0.836 V_{TR} 0.405	*a* 0.223 *d* 0.092	*b* 0.112 *e* 0.122	*c* 1.89 *f* 0.096						
Density 33 lb. per cu. ft.	El. Lt.	5,640	460	240	.....	....	....	....	....	......							6.8	8.75	2.2	7.0	9.2	2.0
Moisture 13.4%	Ult.	6,500	1,920	1,000	17,800	1,080	650	1,050	1,540	500 approx.												
Ash:	$E \times 10^6$	2.18	0.233	0.14	.....	....	....	....	....	......	V_{LR} 0.533 V_{RL} 0.0582	V_{LT} 0.653 V_{TL} 0.0421	V_{RT} 0.656 V_{TR} 0.386	*a* 0.352 *d* 0.214	*b* 0.206 *e* 0.284	*c* 2.47 *f* 0.148						
Density 50 lb. per cu. ft.	El. Lt.	3,800	400	170	.....	....	....	....	....	......							8.9	12.4	3.6	16.0	21.8	4.8
Moisture 13.6%	Ult.	7,300	2,450	2,000	25,400	2,270	1 080	2,000	2,000	600 approx.												
Walnut:	$E \times 10^6$	1.63	0.172	0.092	.....	....	....	....	....	......	V_{LR} 0.0495 V_{RL} 0.052	V_{LT} 0.632 V_{TL} 0.036	V_{RT} 0.718 V_{TR} 0.367	*a* 0.259 *d* 0.140	*b* 0.139 *c* 0.190	*c* 1.82 *f* 0.107						
Density 37 lb. per cu. ft.	El. Lt.	4,200	650	520	.....	....	....	....	....	......							0.1	13.9	3.4	10.2	12.5	3.2
Moisture 11.0%	Ult.	8,480	1,700	1,380	24,000	1,690	1,080	2,000	2,000	500 approx.												

[1] Elastic limit.

for the expression of the relationship between stress and strain in terms of the elastic constants is rather complicated. The use of Mohr's stress and strain circles,* to represent graphically the relation between stress or strain in one direction to stress or strain in any other direction, is a much easier method of evaluating the stresses or strains when the forces do not act along the natural axes of wood.

62. Mohr's Circles of Stress and Strain. The graphical representation of stresses or strains is known as Mohr's circle. Before discussing this method of solving for stresses and strains on any plane due to principal stresses or strains, let us consider the elementary equations representing these stresses and strains. The equations which follow are purely geometrical and are applicable to any material regardless of the elastic constants.

In the simple case where a wood member is subjected to tension only, the normal and shearing stresses on any plane inclined to the axis of the member are as follows:

$$f_n = f_x \cos^2 \theta$$

$$S = \frac{f_x}{2} \sin 2\theta$$

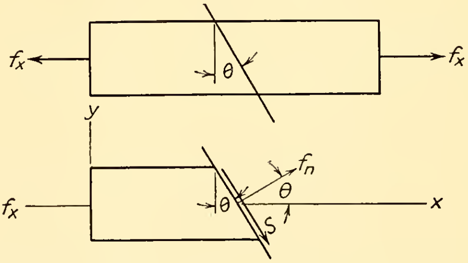

FIG. 18. Stresses on an inclined plane for a member subjected to simple tension.

For the two-dimensional case in which the member is acted on by tension or compression in two perpendicular directions, the normal and shearing stresses become

$$f_n = f_x \cos^2 \theta + f_y \sin^2 \theta$$

$$S = \tfrac{1}{2}(f_x - f_y) \sin 2\theta$$

The equations for the strains may be derived for the general case in which a cubical block is subjected to normal and shearing stresses on two pairs of opposite faces as shown in Fig. 19.

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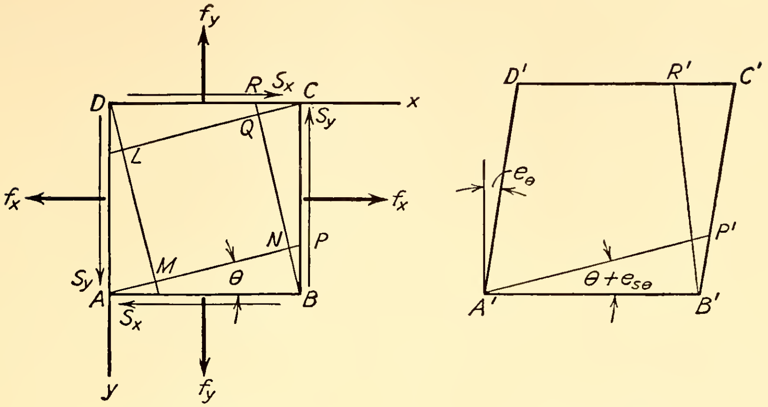
* "The Application of Mohr's Stress and Strain Circles to Wood and Plywood," by C. B. Morris, *Mimeo.* 1317, U. S. Dept. Agr., 1943.



Unstrained

Strained

FIG. 19. Cubical block subjected to normal and shearing stresses on two pairs of opposite faces.

If we consider the original length of AP as L and the extension of AP as e, the unstrained lengths of AB and BP are $L \cos\theta$ and $L \sin\theta$ respectively. After strain the lengths are

$$A'P' = L(1 + e)$$

$$A'B' = L\cos\theta(1 + e_x)$$

$$B'P' = L\sin\theta(1 + e_y)$$

and angle ABP becomes $A'B'P' = (\pi/2) + e_\theta$, where e_θ is the shearing strain.

$$(A'P')^2 = (A'B')^2 + (B'P')^2 - 2A'B' \times B'P' \cos A'B'P'$$

$$L^2(1 + e)^2 = L^2\cos^2\theta(1 + e_x)^2 + L^2\sin^2\theta(1 + e_y)^2 - L^2\sin 2\theta(1 + e_x)(1 + e_y)\sin e_\theta$$

Neglecting terms of the second order in e_1, e_2, and e_θ and simplifying, the above equation becomes

$$e = e_x\cos^2\theta + e_y\sin^2\theta + \frac{e_\theta}{2}\sin 2\theta$$

When the shearing stress on the block is zero and x and y become axes of principal strain, we have

$$e = e_x\cos^2\theta + e_y\sin^2\theta$$

The shear strain in the block $LMNQ$ may be found by denoting the angle PAB after strain as $\theta + e_{s\theta}$, where $e_{s\theta}$ is the increase in the angle

PAB due to the counterclockwise rotation of AP, and proceeding as follows:

$$\tan(\theta + e_{s\theta}) = \frac{B'P' \cos e_\theta}{A'B' + B'P' \sin e_\theta}$$

$$= \frac{BP(1 + e_y)\cos e_\theta}{AB(1 + e_\theta) + BP(1 + e_y)\sin e_\theta}$$

$$= \frac{\tan\theta(1 + e_y)\cos e_\theta}{1 + e_x + (1 + e_y)\sin e_\theta \tan\theta}$$

If we neglect e_x, e_y, and e_θ, $\tan(\theta + e_{s\theta}) = \tan\theta$, and $e_{s\theta}$ is a small quantity of the order of strain. To the first order, we have

$$\frac{\tan\theta + e_{s\theta}}{1 - e_{s\theta}\tan\theta} = \tan\theta\left(\frac{1 + e_y}{1 + e_x + e_\theta \tan\theta}\right)$$

To the same order of approximation, the value of $e_{s\theta}$ becomes

$$e_{s\,\theta} = -\tfrac{1}{2}(e_x - e_y)\sin 2\theta - e_\theta \sin^2\theta$$

The angle $e_{s\theta}$ is the increase, due to strain, in the angle PAB due to the rotation of AP in the counterclockwise direction. Substituting $\theta + (\pi/2)$ for θ in the above expression, we obtain the corresponding rotation of BR:

$$e'_{s\theta} = \tfrac{1}{2}(e_x - e_y)\sin 2\theta - e_\theta \cos^2\theta$$

The shearing strain, e'_θ, in the block $LMNQ$ is given by

$$e'_\theta = e_{s\theta} - e'_{s\theta}$$

Substituting the values of $e_{s\theta}$ and $e'_{s\theta}$ in the above expression, we have for the shear strain

$$e'_\theta = -(e_x - e_y)\sin 2\theta + e_\theta \cos 2\theta$$

If there is no shearing stress on the faces of the block, the last term in the equation for rotational strain, $e_{s\theta}$, and shearing strain, e'_θ, drops out. A comparison of the two equations shows that the rotational strain is one-half the value of the shearing strain.

To illustrate the graphical representation for the condition of two perpendicular stresses, consider Fig. 20.

The abscissas at A and B represent to scale the stresses f_x and f_y parallel to the x and y axes respectively. In order to obtain the stress components on any inclined plane it is necessary to construct a circle with a diameter equal to AB and draw the radius CD making an angle

2θ measured in the counterclockwise direction from point A. From Fig. 20 it is apparent that

$$\overline{OE} = \overline{OC} - \overline{CE} = \tfrac{1}{2}(\overline{OA} + \overline{OB}) - \tfrac{1}{2}(\overline{OB} - \overline{OA}) \cos 2\theta$$

$$\overline{OE} = \frac{f_x + f_y}{2} - \left(\frac{f_y - f_x}{2}\right) \cos 2\theta$$

or, in another form,

$$\overline{OE} = f_x \cos^2 \theta + f_y \sin^2 \theta$$

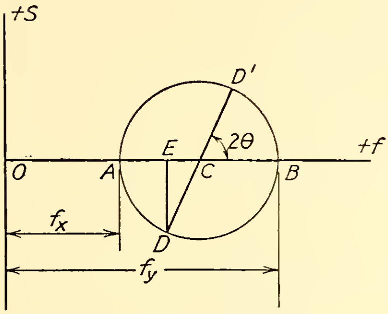


FIG. 20. Mohr's circle of stress.

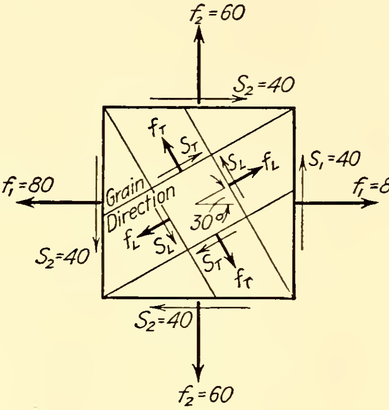


FIG. 21. Block of wood with normal and shearing stresses acting on two faces.

$\overline{OE}$ represents the normal stress f_n on the inclined plane, such that f_n acts at an angle θ with respect to f_x.

$$\overline{DE} = CD \sin 2\theta = \left(\frac{f_y - f_x}{2}\right) \sin 2\theta$$

$\overline{DE}$, which is the ordinate of the point D, represents the shearing stress on the inclined plane.

A similar geometrical analysis may be obtained for the relations between the strains.

In the application of the graphical method to the analysis of stresses and strains in wood it must be remembered that the principal axes of strain are not always parallel to the principal axes of stress, and the angle θ in the formulas for stress and strain is not necessarily the same angle in both formulas.

As an example, consider the two-dimensional case illustrated in Fig. 21. The stresses are in pounds per square inch. The problem is to determine the stresses and strains along and across the grain. Once the stresses are determined the strains may be found from the following relations, and the strain circle may then be drawn.

$$e_L = \frac{f_L}{E_L} - \frac{V_{TL} f_T}{E_T}$$

$$e_T = \frac{f_T}{E_T} - \frac{V_{LT} f_L}{E_L}$$

$$e_{SL} = \frac{S_{LT}}{2G_{LT}}$$

where e_{SL} = rotational strain which is one-half the shear strain.

If f_1, f_2, S_1, and S_2 are drawn to scale and the circle of stress completed, it will be found that the angle 2θ is equal to 105° as illustrated in Fig. 22. θ is the angle between the line of action of the principal stress and f_2. With this angle known, the angle θ_T between the principal stress and f_T is readily obtained from Fig. 23.

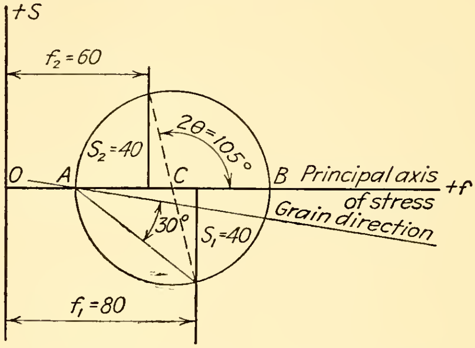

+S
f₂ = 60
2θ = 105°
S₂ = 40
O
A
C
B Principal axis of stress
+f
Grain direction
30°
S₁ = 40
f₁ = 80

Fig. 22. Circle of stress for normal and shearing stresses on faces of block.

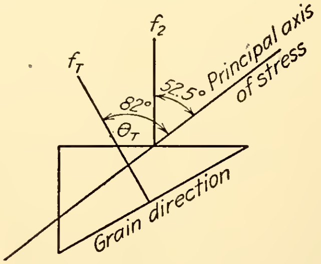

f₂
f_T
82°
52.5°
θ_T
Principal axis of stress
Grain direction

Fig. 23. Location of principal axis of stress.

By laying off an angle $2\theta_T$, measured counterclockwise around the line AB, a point on the stress circle is found, and the horizontal distance from O to the ordinate through this point is f_T. The stress f_L acting at 90° to f_T and the shearing stresses S_L and S_T are represented as shown in Fig. 24.

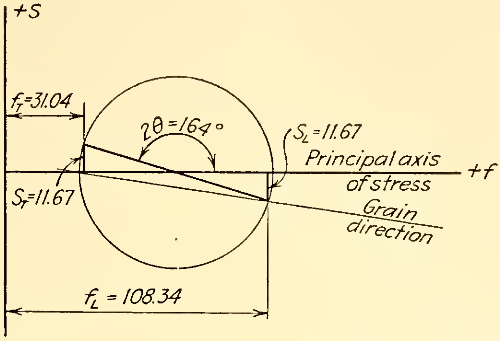


FIG. 24. Determination of stresses along and across the grain.

All the stresses on the inner block which are parallel to the natural axes of the wood have been determined, and the strains may be determined from those stresses.

If the wood is spruce, the following relations are found from Table 11:

$$E_T = 0.036E_L$$

$$G_{LT} = 0.037E_L$$

$$V_{TL} = 0.0194$$

$$V_{LT} = 0.539$$

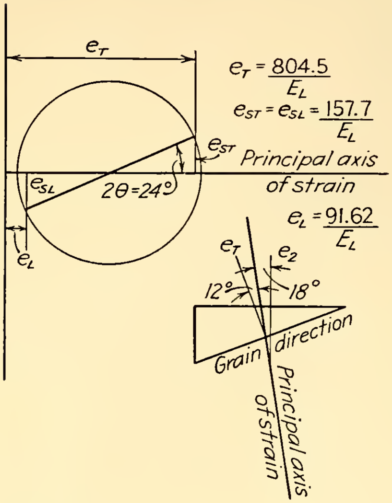
By substituting in the equations previously given for strains in term of stresses we obtain

$$e_L = \frac{108.34}{E_L} - \frac{0.0194 \times 31.04}{0.036E_L} = \frac{91.62}{E_L}$$

$$e_T = \frac{31.04}{0.036E_L} - \frac{0.539 \times 108.34}{E_L} = \frac{804.5}{E_L}$$

$$e_{SL} = \frac{11.67}{2 \times 0.037E_L} = \frac{157.7}{E_L}$$

In order to avoid the use of exceedingly small numbers, the strains are given in terms of the modulus of elasticity along the grain. By using these strains in plotting the strain circle, the direction of the principal axis of strain may be determined, and the strains along the axis of the applied force are easily obtained as illustrated in Figs. 25 and 26.


e_T
$e_T = \frac{804.5}{E_L}$
$e_{ST} = e_{SL} = \frac{157.7}{E_L}$
e_{ST}
Principal axis of strain
e_{SL}
$2\theta = 24°$
$e_L = \frac{91.62}{E_L}$
e_L
e_T
e_2
12°
18°
Grain direction
Principal axis of strain

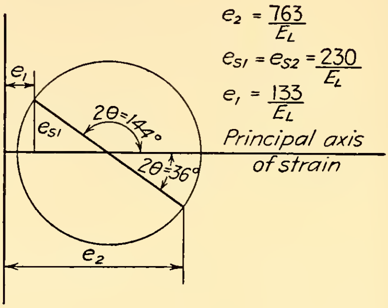

$e_2 = \frac{763}{E_L}$
$e_{S1} = e_{S2} = \frac{230}{E_L}$
$e_1 = \frac{133}{E_L}$
Principal axis of strain
e_1
e_{S1}
$2\theta = 144°$
$2\theta = 36°$
e_2

FIG. 25. Circle of strain and location of principal axis of strain.

FIG. 26. Determination of strains along axis of stresses on face of wood block.

63. Form Factors. Since the strength of wood in tension and compression along the grain is very different, being much greater in tension, it probably seems unreasonable that wood beams should behave in a manner similar to homogeneous or isotropic materials. Furthermore, test data show that the computed fiber stress at the elastic limit in bending is far greater than the fiber stress at the elastic limit in compression parallel to the grain. It might be expected then that the design of a wood beam in bending should be governed by the allowable unit stress in compression parallel to the grain. That this is not true has been proved by a multitude of tests. The general theory advanced to explain the apparently higher compressive stress existing in a beam is that the minute fibers are not uniformly stressed, and those at or near the neutral axis, which carry the least stress, will not buckle but will add support to the extreme fibers and thus make it possible for them to carry a higher load. In direct compression parallel to the grain the individual wood fibers act as Euler columns, and when all are stressed alike they offer little support to each other.

The test results from which basic stresses for clear specimens are derived were based on tests of pieces 2 by 2 in. in cross section. Additional tests on structural sizes have shown that, as the height of a rectangular beam is increased, the modulus of rupture decreases slightly. In a comparison of beams of different heights this decrease should be considered by adding a form factor as a multiplier in the usual beam formula so that the resisting moment M is equal to FfS where

F = form factor.
f = unit working stress in bending.
S = section modulus.

An empirical formula for the form factor based on tests of beams up to 12 in. deep is as follows:

$$F = 1 - 0.07\left(\sqrt{\frac{h}{2}} - 1\right)$$

where h = depth of the beam.

This variation in strength with the depth of a beam is in agreement with the beam theory advanced in the previous paragraphs. In a deep beam the stress decrease from the outer fiber to the neutral axis is less rapid than in a shallow beam, and the fibers adjacent to the surface are less able to add support to the highly stressed extreme fibers.

In the tests at the Forest Products Laboratory on beams of structural sizes, a form factor of 90 per cent was used in determining working stresses in extreme fiber in bending. This means that it is not necessary to include a form factor when designing rectangular beams and using the working stresses given in Table 8.

A square placed in position with a diagonal vertical will sustain the same load as a beam of the same cross section loaded on one side. Since the moment of inertia of a square about a neutral axis perpendicular to its sides is the same as the moment of inertia about a diagonal and since the bending moment a beam will carry is directly proportional to I and inversely proportional to c, it is necessary to use a form factor in the usual beam formula. The distance c from the neutral axis to the extreme fiber of a square beam with a diagonal vertical is $\sqrt{2}$ times as great as that for a beam with sides vertical. Both beams will carry the same load, and a factor equal to $\sqrt{2}$ or 1.414 must be applied to the usual beam formula in calculating the resisting moment of a square beam with a diagonal vertical so that the formula becomes $M = 1.414fI/c$.

A similar condition exists for a beam of circular section. Tests have shown that a beam of circular section will sustain practically the same load as a square beam of equal area. However, the section modulus I/c for a square is approximately 1.18 times as great as that for a circle of equal area. For beams of circular section a form factor of 1.18 should be added to the usual formula so that $M = 1.18fI/c$.

In an I-beam a different situation exists because only those fibers in a width which are equal to the width of the web get the complete supporting action similar to that in a solid beam. The amount of support for the fibers outside the web is limited to the depth of the compression flange. It is apparent, then, that an I-beam is weaker than a solid beam of the same height and section modulus. The amount of reinforcement or supporting action received by the extreme compressive fibers from those less stressed is difficult to evaluate, but suitable form factors have been developed from a series of tests.* The results of these tests have been represented by simple mathematical curves as illustrated in Fig. 27.

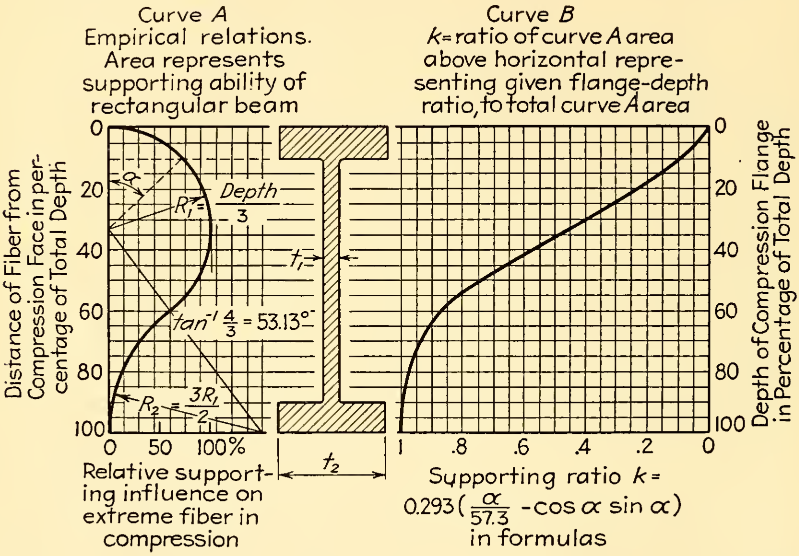


FIG. 27. Curves showing empirical relations in stress factor theory based on assumption that less stressed fibers lend support to higher stressed fibers.

The abscissas in curve *A* represent the relative supporting influence of all fibers, and the total area under the curve represents the total support received by the extreme compressive fiber of a solid beam. The area to a depth equal to the compression flange as compared with the total area represents the relative support of the extreme fiber in the flange of an I-beam or box beam exclusive of that portion which may be considered the web extended through to the top.

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* "Form Factors of Beams Subjected to Transverse Loading Only," by J. A. Newlin and G. W. Trayer, *Tech. Bull.* 1310, U. S. Dept. Agr., 1941.

If the radius R_1 in Fig. 27 is assumed to be unity, the total area A between the curve and the vertical axis is

$$A = \frac{1}{2}\left[\frac{143.13^\circ}{57.3} + \left(\frac{3}{2} \times 2\right) - \frac{53.13}{57.3}\left(\frac{3}{2}\right)^2\right]$$

The area A' of the portion above the dotted line representing the flange-depth ratio of an I-section or box section is

$$A' = \frac{1}{2}\left(\frac{\alpha}{57.3} - \sin\alpha\cos\alpha\right)$$

The above formulas are applicable when the depth of the compression flange is not more than 60 per cent of the total depth of the beam. Within these limits α is the angle whose versed sine is 3 × depth of compression flange/depth of beam.

If the width of the flange of an I- or box-beam is t_2 and the width of the web is t_1, the supporting ability of the compression flange would be $A'/A[(t_2 - t_1)/t_2]$ times the supporting ability of the rectangle t_2 wide. The supporting ability of the web will be t_1/t_2 times the reinforcement of a rectangle t_2 wide.

A comparison of the elastic limit of a rectangular section in compression parallel to the grain to that in bending will show that the lateral support given to the more stressed fibers, by those less stressed, increases the fiber stress at the elastic limit in flexure over that in direct compression by about 72 per cent. The increase, then, of fiber stress at elastic limit for an I- or box-section may be expressed as

$$0.72\left[\frac{A'}{A}\left(\frac{t_2 - t_1}{t_2}\right) + \frac{t_1}{t_2}\right]$$

The ratio of the elastic-limit stress in bending to that in direct compression will be 1 plus the above quantity divided by 1.72. Consequently, the form factor within the elastic limit for an I- or box-section becomes

$$F = 0.58 + 0.42\left[0.293\left(\frac{\alpha}{57.3} - \sin\alpha\cos\alpha\right)\frac{t_2 - t_1}{t_2} + \frac{t_1}{t_2}\right]$$

The quantity 0.293 $[(\alpha/57.3) - \sin\alpha\cos\alpha]$ is the ratio of A'/A and may be represented graphically as K in curve B of Fig. 27. The form factor then becomes

$$F = 0.58 + 0.42\left[K\left(\frac{t_2 - t_1}{t_2}\right) + \frac{t_1}{t_2}\right]$$

where K depends upon the depth of the compression flange in percentage of the total depth.

By substituting 0.50 for both 0.58 and 0.42, the form factor beyond the elastic limit or for the modulus of rupture becomes

$$F = 0.50 + 0.50\left[K\left(\frac{t_2 - t_1}{t_2}\right) + \frac{t_1}{t_2}\right]$$

64. Horizontal Shear. Tests * at the Forest Products Laboratory have shown that the usual formula without correction for calculating horizontal shear in beams is overconservative and does not provide for economical design. These tests indicate that beams containing checks or shakes at or near the neutral axis act partly as two beams and partly as a unit. Since the upper and lower halves of the beam act to some extent as individual beams, some relief of the mean shearing stress at the neutral axis results.

The results of over 200 tests, in which loads causing shearing failure were observed on built-up artificially checked beams ¾ in. by 1½ in. to 8 in. by 16 in. in cross section, and an approximate mathematical analysis have shown that the reaction R is made up of two factors, namely, the independent action of the upper and lower halves of the beam and the single-beam action of the entire cross section. The equation representing the reaction is expressed by

$$R = B + \frac{A}{a^2}$$

B represents the single-beam portion of the reaction and is equal to $\frac{2}{3}Jbh$, where J is the mean shearing stress at the neutral axis over the entire width of the beam, b the width, and h the depth; a is the distance from the load to the nearest support. A is the independent action of the two halves of the beam and is not associated with the shear at the neutral axis. It is a quantity determined chiefly by the dimensions of the beam, by longitudinal Young's modulus, and by the mean longitudinal displacement on the lower face of the upper half of the beam at points immediately over the support.

The values of the two-beam and single-beam portions of the reaction at the instant of failure were determined by testing a large number of beams and by combining the results of tests of carefully matched beams loaded to failure, the concentrated loads being applied at different points. By entering the results of each pair of tests in the equation for the reaction, two equations in A and B were obtained, and their values could be found.

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* "New Method for Calculating Horizontal Shear in Wooden Beams," by J. A. Newlin, G. E. Heck, and H. W. March, *Eng. News-Record*, Vol. 110, 1938.

The distribution of shearing stresses is shown in Fig. 28.

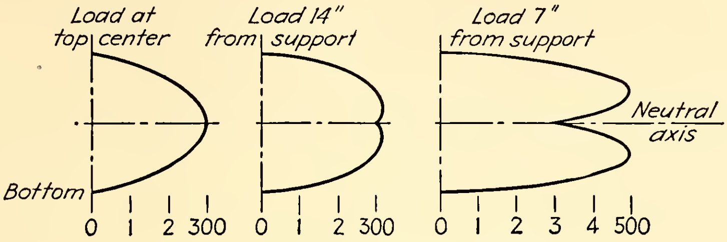

Fig. 28. Distribution of shearing stresses.

From the test data and mathematical treatment, the following recommendations have been developed for use in design:

1. Use the ordinary shear formula and the working stresses for timber.
2. Take into consideration any relief to the beam resulting from load being distributed to adjacent parallel beams by flooring or other members.
3. Neglect all loads within the height of the beam from both supports.
4. If there are any moving loads, place the large one at three times the depth of the beam from the support or at the quarter point, whichever is closer to the support. All other loads should be kept in their normal positions.
5. Treat all other loads in the usual manner.
6. If a timber does not qualify for shear resistance under the foregoing procedure, determine the end shear from the following formulas:

For a concentrated load

$$V = \frac{10P(l - x)\left(\frac{x}{h}\right)^2}{9l\left[2 + \left(\frac{x}{h}\right)^2\right]}$$

where V is the shear or reaction to be used as due to the load P, L the span in inches, x the distance in inches from the reaction to the load P. and h the height of the beam in inches.

For a uniformly distributed load

$$V = \frac{W}{2}\left(1 - \frac{2h}{l}\right)$$

65. Direct Compression. For short columns in which the ratio of the length to the least dimension l/d is less than 11, failure is due to crushing, and the working stress for the column is equal to the safe unit stress in compression parallel to the grain.

Intermediate columns having an l/d ratio between 11 and K fail by a combination of crushing and lateral buckling. The significance of K and the working stress are explained in the following paragraphs.

Long columns are those with an l/d ratio greater than K and, for this type of column, failure is by lateral buckling, and the column behaves essentially as a Euler column.

Tests * of large timber columns at the Forest Products Laboratory have verified the existence of a formula for intermediate columns of wood which is different from any formulas used for other materials. The Euler formula is recommended as the best law governing the strength of long columns of uniform cross section within the elastic limit of the material. The general form of this formula is

$$\frac{P}{A} = u\frac{\pi^2 E}{\left(\frac{l}{r}\right)^2}$$

where P = maximum load on the column in pounds.
A = cross-sectional area in square inches.
E = modulus of elasticity in pounds per square inch.
l = length in inches.
r = radius of gyration in inches.
l/r = slenderness ratio.
u = factor depending on end conditions (for pin ends $u = 1$).

In wooden columns the curve representing the load which a column will carry for different slenderness ratios between the elastic limit to the point of maximum stress is a smooth curve. This curve becomes tangent to the Euler curve at a P/A value equal to the elastic limit stress and is parabolic with its vertex at zero l/r at the point of maximum stress. The curve represents the values of P/A for intermediate columns, and the general form of the equation of the parabolic curve is

$$\frac{P}{A} = c\left[1 - \left(\frac{c-f}{f}\right)\left(\frac{\frac{l}{r}}{\frac{l}{r'}}\right)^{2f/(c-f)}\right]$$

---

* "Tests of Large Timber Columns and Presentation of the Forest Products Laboratory Column Formula," by J. A. Newlin and J. M. Gahagan, *Tech. Bull.* 167, U. S. Dept. Agr., 1930.

where c = maximum crushing strength in pounds per square inch.
f = fiber stress at elastic limit in pounds per square inch
l/r' = slenderness ratio of the column when $P/A = f$.

For columns of rectangular section the above formula may be written

$$\frac{P}{A} = c\left[1 - \left(\frac{c-f}{c}\right)\left(\frac{\frac{l}{d}}{\frac{l}{d'}}\right)^{2f/(c-f)}\right]$$

where d = least dimension of column in inches = $r\sqrt{12}$.
l/d = ratio of length to least dimension.
l/d' = slenderness ratio when $P/A = f$. It is the l/d ratio at the point of tangency between the parabola and the Euler curve.

When the elastic limit of the column is four-fifths of the maximum crushing strength and if l/d' is replaced by K_1 the parabolic equation becomes

$$\frac{P}{A} = c\left[1 - \frac{1}{5}\left(\frac{l}{K_1 d}\right)^8\right]$$

When the elastic limit of the column is two-thirds of the maximum crushing strength and l/d' is replaced by K the equation becomes a fourth-power parabolic formula:

$$\frac{P}{A} = c\left[1 - \frac{1}{3}\left(\frac{l}{Kd}\right)^4\right]$$

The above equation is recommended for the design of intermediate wood columns by replacing the maximum crushing strength c with the allowable unit stress in compression parallel to the grain. K is a value which depends upon the modulus of elasticity E of the species and the allowable stress in compression parallel to the grain. Its value may be found by substituting the value of $P/A = \frac{2}{3}c$ in the Euler formula:

$$\frac{P}{A} = \frac{\pi^2 E}{\left(\frac{l/d}{\sqrt{12}}\right)^2} = \frac{2}{3}c$$

Introducing a factor of safety of 3, letting $l/d = K$, and solving for K, we obtain

$$K = \frac{\pi}{2}\sqrt{\frac{E}{6c}} = 0.64\sqrt{\frac{E}{c}}$$

The recent increase in working stresses adds 20 per cent to the value of P/A determined from the Euler formula, and K becomes

$$K = \frac{\pi}{2}\sqrt{\frac{E}{5c}} = 0.702\sqrt{\frac{E}{c}}$$

A comparison of the Forest Products fourth-power parabolic formula with other formulas is shown in Chart 4.

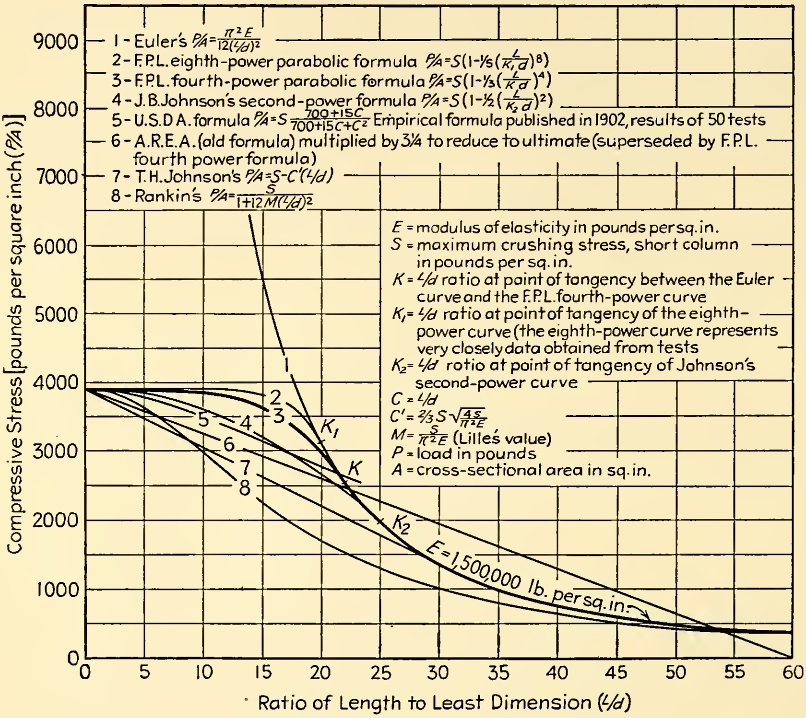

CHART 4. Column formulas and the resultant curves.

66. Combined Stresses. Since wood does not have the same unit stress in bending as in direct compression, compression members subjected to an eccentric load or a side load require special formulas for the calculation of the maximum direct compressive loads. These formulas have been developed,* and their derivation is given in the following paragraphs.

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* "Formulas for Columns with Side Load and Eccentricity," by J. A. Newlin, *Building Standards Monthly*, December, 1940.

Consider the column shown in Fig. 29 with end loads W. The column is assumed to have an initial deflection equal to e_1, and the total deflection at the center of the column is represented as y_o. If the original shape of the column with an initial curvature is considered as a sine curve, the equation representing this curvature may be expressed as *

$$y_1 = e_1 \cos \frac{\pi x}{L}$$

The bending moment at K is $M = -Wy$, and the change of curvature produced by the bending moment is

$$\frac{d^2}{dx^2}(y - y_1) = \frac{M}{EI} = -\frac{Wy}{EI}$$

The initial curvature is represented by the equation

$$\frac{d^2 y_1}{dx^2} = -\frac{\pi^2 e_1}{L^2} \cos \frac{\pi}{L} x$$

When the above value is inserted in the equation for the change of curvature due to the bending moment, the following result is obtained:

$$\frac{d^2 y}{dx^2} + \frac{2e_1}{L^2} \cos \frac{\pi x}{L} + \frac{W}{EI} y = 0$$

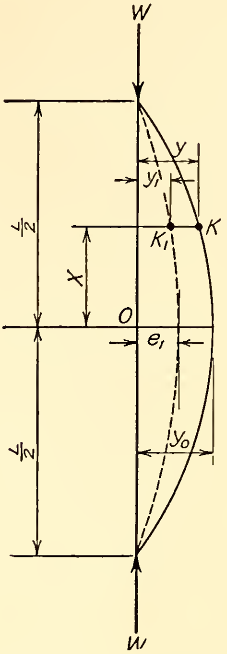

FIG. 29. Column with end loads and initial curvature.

The solution of the foregoing equation gives the deflection y as follows:

$$y = \frac{\pi^2 e_1}{\pi^2 - \frac{WL^2}{EI}} \cos \frac{\pi x}{L} = \frac{e_1}{1 - \frac{WL^2}{\pi^2 EI}} \cos \frac{\pi x}{L}$$

The maximum value of y occurs when $x = 0$.

$$y_o = \frac{e_1}{1 - \frac{WL^2}{\pi^2 EI}}$$

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* "Materials and Structures," by E. H. Salmon, Vol. I, Longmans, Green and Co., 1931.

If we let P be the critical Euler load such that $P = \pi^2 EI/L^2$,

$$y_o = \frac{P}{P - W} e_1$$

This may also be written

$$\frac{e_1}{y_o} = 1 - \frac{W}{P}$$

By using the above equation it is possible to derive formulas for the direct compressive stress for columns with any combination of end loads, side loads, and eccentricity.

SYMBOLS USED

A = cross-sectional area of the column in square inches.
I = moment of inertia of the cross section in inches ^{4}.
c = distance from the neutral axis to the extreme fiber on the concave side of the bent column in inches.
d_b = side of a rectangular column in the direction of the side loads in inches.
l = length of span or column in inches.
e = eccentricity in inches.
f = safe stress in bending in pounds per square inch.
S_b = M/S = unit bending stress due to side load in pounds per square inch.
k = ratio of side load bending stress to unit compressive stress due to end load.
c_b = safe stress as a centrally loaded column.
rS_b = stress in a beam bent to a sinusoidal curve to produce the same maximum deflection as is produced by the given side loads.
Q = allowable stress in compression parallel to the grain that is permitted for the column, when side and/or eccentric load are combined with end load, in pounds per square inch.
W = end load on column in pounds.
P = critical Euler load in pounds.
y_o = deflection of column when all loads are on the column.
e_1 = deflection of a bent column without the end load.

If we transpose W/P in the last equation and consider that the original curvature of the column is due to a side load such that it produces a sinusoidal curve, e_1 and y_o can be replaced by bending stress, and since W and P are loads on the same column, and they may be replaced by the stresses Q and c_b. The equation then becomes

$$\frac{S_b}{f_1} + \frac{Q}{c_b} = 1$$

In the above equation, f_1 is the bending stress due to both end load and side load, and for Q to be a maximum f_1 must be a maximum. If we assume that the total stress cannot exceed the safe bending stress, $f_1 + Q = f$, and this will give a maximum value of Q. Substituting for f_1 in the equation, we have

$$\frac{S_b}{f - Q} + \frac{Q}{c_b} = 1$$

If the side load is proportional to the end load and is such as to produce a sinusoidal curve, the equation becomes

$$\frac{kQ}{f - Q} + \frac{Q}{c_b} = 1$$

If the column has an initial curvature without stress such that e_1 represents the maximum deflection, the total bending stress is proportional to $y_o - e_1$. If $ke_1 = S_b$, $k(y_o - e_1) + Q = f$ for a maximum value of Q.

$$\frac{ke_1}{ky_o} = \frac{S_b}{f - Q + S_b}$$

and the equation becomes

$$\frac{S_b}{f - Q + S_b} + \frac{Q}{c_b} = 1$$

When the column is eccentrically loaded, it was found by trial that e_1 could be replaced by $5e/4$ in the above equation. The values of Q then obtained check quite closely those calculated by use of the secant formula. The formula for eccentric loading then becomes

$$\frac{c_b\left(\dfrac{5ecA}{4I}\right)}{f - Q + C_b\left(\dfrac{5ecA}{4I}\right)} + \frac{Q}{c_b} = 1$$

By solving the foregoing equations for Q, we shall have formulas for the maximum permissible stress on columns eccentrically loaded or for any combination of end load, side load, and eccentricity. (See Chapter VI.)

It must be remembered that the derivation of these formulas is based on several assumptions.

1. The stresses which cause a given deflection as a sinusoidal curve are the same as those for a beam with a uniform load.
2. For a single concentrated side load the stress under the load can be used, regardless of the position of the load with reference to the length of the column.
3. The stress to use with a system of side loads is the maximum stress due to the system. (With large side loads near each end a slight error on the side of overload will occur.)
4. The equations can be used for solving for Q with rectangular wood columns with an l/d ratio of 20 or more provided that, for columns between $l/d = 20$ and $l/d = K$, the c_b is the stress for a centrally loaded column of intermediate length, using the Forest Products Laboratory fourth-power parabolic formula.
5. For columns with an l/d ratio of 11 or less the stress due to deflection of the column may be neglected.
6. Between l/d ratios of 11 and 20 the stress may be assumed to vary as a straight line.

67. Wood Joints. The mathematical solution for the stress distribution in wood joints is an extremely difficult problem and has not been solved theoretically. The many factors governing the ultimate or safe loads for joints with various types of fastenings makes it impossible to derive equations by any method other than actual test results. In general, the variables which determine the resistance to load in wood joints are the specific gravity of the wood, the species, the moisture content, the type and size of fastening, the direction of the load with respect to the grain direction of the wood, the direction of the load with respect to the axis of the fastening, the bending of the fastening, and the type and size of splice plates or cleat used. The general equations for determining the loads on wood joints for various fastenings and the factors governing their derivation are given in Chapter V.

CHAPTER V

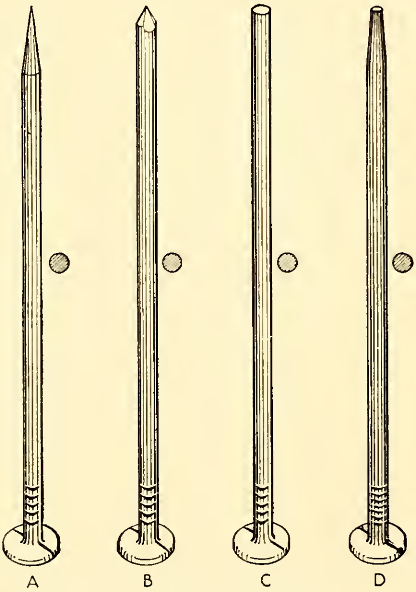
FASTENINGS

68. General. The formulas for computing the allowable loads for all fastenings, except connectors, given herein are based on permanent or long-time loads. For short-time loads, such as snow, wind, and earthquake loads, the equations should be modified in accordance with the provisions set up for working stresses in Chapter III.

Through hundreds of tests the Forest Products Laboratory has developed safe loads for all types of wood fastenings. The material following is a presentation of the different types of fastenings, the formulas, and their derivation, for calculating safe loads, and the methods governing their use in design.

NAILS AND SPIKES

69. Types. Several types of nail points and nail shanks are illustrated in Figs. 30 and 31. In general, nails with long sharp points have a higher holding power than those with common points; but if the species of wood has a tendency to split, a sharp point will accentuate this tendency, and the holding power will be reduced. A blunt point reduces splitting but causes destruction of the wood fibers when it is driven into the wood; this reduces the holding power below that of a common nail. A blunt point tapered at the end will not cause splitting as readily as the common nail and is equal to the common nail in holding power when used in the heavier woods, but its resistance to

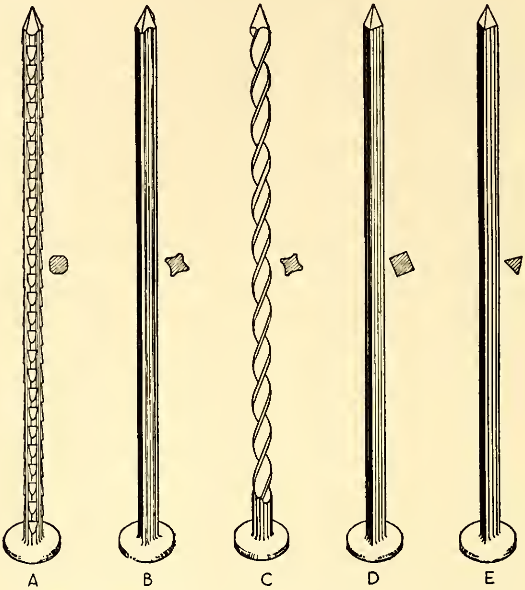


Courtesy Timber Engineers' Handbook, Edited by H. J. Hansen

FIG. 30. Types of nail points: (*a*) long and sharp, (*b*) common, (*c*) blunt, and (*d*) blunt tapered.

withdrawal is less than that of the common nail in woods of lesser density.

All types of nail shanks, except the barbed shank, give higher resistance to withdrawal than the plain nail. This is true under all conditions of moisture content, and the barbed nail will give higher resistance than a plain nail when it is driven into green wood that is then allowed to season.



Courtesy Timber Engineers' Handbook, Edited by H. J. Hansen

FIG. 31. Types of nail shanks: (*a*) barbed, (*b*) longitudinally grooved, (*c*) spirally grooved, (*d*) square, and (*e*) triangular.

If a nail is given a cement coating of good quality, its resistance to withdrawal will increase, immediately after driving, by 85 to 100 per cent over plain nails. This is true only for the lighter woods, and in approximately one month only about one-half of this increase will remain. Zinc coatings have no particular advantage except to reduce or prevent corrosion. If the coating is evenly applied it may increase the holding power, but any irregularities of the coating may actually reduce the holding power.

Common wire spikes are nothing more than large nails and are made in lengths from 3 to 12 in. Usually they have either a diamond or a chisel point.

70. Sizes. The sizes for various nails and spikes are given in Table 12. For corresponding lengths, spikes have larger diameters than nails, and beyond the sixtypenny size spikes are usually designated by inches of length.

Table 12. Sizes of Nails and Spikes

[American Steel and Wire Co.]

Gage	1	2	3	4	5	6	7	8	9	10	10¼
Diameter	0.2830	0.2625	0.2437	0.2253	0.2070	0.1920	0.1770	0.1620	0.1483	0.1350	0.1314

Gage	10½	11	11½	12	12½	13	14	14½	15	15½	16
Diameter	0.1278	0.1205	0.1130	0.1055	0.0985	0.0915	0.0800	0.0760	0.0720	0.0673	0.0625

Name	Size	2d	3d	4d	5d	6d	7d	8d	9d	10d	12d	16d	20d	30d	40d	50d	60d
	Length	1 in.	1¼	1½	1¾	2	2¼	2½	2¾	3	3¼	3½	4	4½	5	5½	6
Common	Gage	15	14	12½	12½	11½	11½	10¼	10¼	9	9	8	6	5	4	3	2
	No. per lb.[1]	876	568	316	271	181	161	106	96	69	63	49	31	24	18	15	11
Siding	Gage	15½	14½	14	14	12½	12½	11½	11½	10½							
	No. per lb.	1010	635	473	406	236	210	145	132	94							
Finishing	Gage	16½	15½	15	15	13	13	12½	12½	11½	11½	11	10				
	No. per lb.	1351	807	584	500	309	238	189	172	121	113	90	62				
Flooring	Gage	...	...	...	...	11	11	10	10	9	8	7	6				
	No. per lb.	...	...	...	...	157	139	99	90	69	54	43	31				
Fence	Gage	...	...	...	10	10	9	9	8	7	6	5					
	No. per lb.	...	...	...	142	124	92	82	62	50	40	30					

Standard barbed roofing nails	Size and length	¾ in.	⅞	1	1⅛	1¼	1⅜	1½	1¾	2
	Gage	13	12	12	12	11	11	10	10	9
	No. per lb.[1]	714	469	411	365	251	230	176	151	103

Round spikes	Size	10d	12d	16d	20d	30d	40d	50d	60d	7 in.	8	9	10	12
Flathead	Length	3 in.	3¼	3½	4	4½	5	5½	6	7	8	9	10	12
Diamond point	Gage	6	6	5	4	3	2	1	1	5/16 in.	⅜	⅜	⅜	⅜
	No. per lb.[1]	41	38	30	23	17	13	10	9	7	4	3½	3	2½

[1] Approximate.

71. Withdrawal Resistance. The withdrawal resistance for joints with nails or spikes depends upon the direction of the penetration of the fastening with respect to the grain direction, the depth of penetration of the fastening, the diameter of the fastening, and the species, specific gravity, and moisture content of the wood. From test results a general formula for the ultimate withdrawal resistance for nails and spikes driven into the side grain of seasoned wood is expressed:

$$P = KG^nD$$

where P = ultimate load in pounds per lineal inch of penetration.
K = constant depending upon the species.
G = specific gravity based on oven-dry wood.
n = power of specific gravity.
D = diameter of nail or spike in inches.

From the above equation it is apparent that the withdrawal load per lineal inch of penetration of the fastening is directly proportional to the species used, to some power of the specific gravity, and to the diameter of the nail or spike, provided that the fastening is driven into the side grain of seasoned wood. For safe working loads with a factor of safety of approximately 6, the constant K for nails and spikes is usually taken as 1380 and the power of the specific gravity as $5/2$ so that the safe withdrawal load becomes

$$P = 1380G^{5/2}D$$

Certain species will give somewhat higher values than those found by using the general equation above, and others will fall below the equation value. For longleaf and shortleaf pine the Forest Products Laboratory recommends using 80 per cent of the equation value for sixpenny and twelvepenny nails, 70 per cent for the twentypenny and larger sizes, and 75 per cent for sixteenpenny nails. For Sitka spruce the values given by the formula should be increased 10 per cent, and for northern white pine an increase of 20 per cent is recommended.

Table 13. Specific Gravity of Important Structural Woods[1]

Species	*Specific Gravity*	*Species*	*Specific Gravity*
Alder, red	0.43	Gum, tupelo	0.52
Ash, black	0.53	Hackberry	0.56
Ash, white	0.64	Hemlock, eastern	0.43
Aspen	0.40	Hemlock, western	0.44
Basswood	0.40	Hickory	0.72
Birch, yellow	0.66	Larch, western	0.59
Buckeye, yellow	0.38	Locust, black	0.71
Butternut	0.40	Madrone	0.69
Cedar, northern white	0.32	Magnolia, evergreen	0.53
Cedar, Port Orford	0.44	Maple, hard	0.68
Cedar, western red	0.34	Maple, soft	0.51
Cherry, black	0.53	Oak	0.68
Chestnut	0.45	Pine, northern white	0.37
Cottonwood, eastern	0.43	Pine, ponderosa	0.42
Cucumber, magnolia	0.52	Pine, southern yellow	0.58
Cypress, southern	0.48	Poplar	0.43
Douglas fir	0.51	Redwood	0.39
Elm, American	0.55	Spruce	0.39
Elm, rock	0.66	Sycamore	0.54
Fir, commercial white	0.42	Tamarack	0.56
Gum, black	0.55	Walnut, black	0.56
Gum, red	0.53	Willow, black	0.41

[1] Based on volume and weight when oven-dry.

When seasoning takes place after nails are driven into green wood their holding power is generally reduced, and one-fourth of the value obtained from the formula should be used for the safe withdrawal load. In an important structural joint that has been loaded before seasoning takes place, simply resetting the old nails is not sufficient; additional nails should be added to reinforce the joint.

The withdrawal resistance decreases as the angle between the grain and direction of driving decreases. In the softer woods the holding power of nails driven into the end of the piece drops to 75 or 50 per cent of the value obtained from the preceding equation.

72. Lateral Resistance. The general equation for the safe lateral load for nails and spikes driven into the side grain of seasoned lumber is $P = KD^{3/2}$.

P = safe lateral load in pounds per nail
K = a constant varying with species
D = diameter of nail in inches

The equations recommended for various species (Table 14) are based on a factor of safety of 1.6 for proportional-limit loads and about 6 for maximum loads.

The equations are based on a penetration, into the wood member holding the point, of two-thirds the length of the nail for softwoods and one-half for hardwoods. For a metal-to-timber connection, the safe lateral resistance determined from the equations given in the tabulation may be increased 25 per cent. If nails are driven into unseasoned wood, which will remain wet or will be loaded before seasoning, the equation values should be reduced by 25 per cent.

For nails or spikes driven into the end grain of wood the equation values should be reduced by one-third.

73. Illustrative Problems. (*a*) Find the withdrawal resistance per lineal inch of penetration for a 12d nail and a spike $3/8$ in. in diameter when driven perpendicular to the grain in seasoned southern pine.

$$P = 1380G^{2\frac{1}{2}}D$$

$$\text{Nail:}\quad P = 1380 \times 0.58^{2\frac{1}{2}} \times 0.1483 \times 0.80 = 41.9 \text{ lb.}$$

$$\text{Spike:}\quad P = 1380 \times 0.58^{2\frac{1}{2}} \times 0.375 \times 0.70 = 92.6 \text{ lb.}$$

(*b*) Find the number and size of spikes to carry a lateral load of 3000 lb. between a 3-in. and an 8-in. piece of seasoned southern pine.

The required penetration of the spike is two-thirds of its length. Select a spike 8 in. long, and check this length for the required penetration, which is $2/3 \times 8$ or 5.33. This result plus the thickness of the 3-in. piece is 7.95 in., and

Table 14. Equation for Computing Safe Lateral Resistance for Common Wire Nails Driven Perpendicular to the Grain of Wood at 15 Per Cent Moisture Content Expressed in Pounds per Nail

[Wood Handbook, U. S. Dept. Agr., 1935]

Species of Wood	*Equation*
Softwoods:	
Cedar, northern and southern white	$P = 1080D^{3/2}$
Fir, balsam and commercial white	
Hemlock, eastern	
Pine, lodgepole, ponderosa, sugar, northern white, and western white	
Spruce, Englemann, red, Sitka, and white	
Cedar, Alaska, incense, Port Orford, and western red	$P = 1350D^{3/2}$
Cedar, eastern red	
Cypress, southern	
Douglas fir (Rocky Mountain region)	
Hemlock, western	
Pine, Norway	
Redwood	
Tamarack	
Douglas fir (coast region)	$P = 1650D^{3/2}$
Larch, western	
Pine, southern yellow	
Hardwoods:	
Aspen and largetooth aspen	$P = 1080D^{3/2}$
Basswood	
Butternut	
Chestnut	
Cottonwood, black and eastern	
Poplar, yellow	
Alder, red	$P = 1500D^{3/2}$
Ash, black	
Birch, paper	
Elm, American and slippery	
Gum, black, red, and tupelo	
Hackberry	
Magnolia, cucumber	
Magnolia, evergreen	
Maple, bigleaf	
Maple (soft), red and silver	
Sugarberry	
Sycamore	
Ash, commercial white	$P = 2040D^{3/2}$
Ash, Oregon	
Beech	
Birch, sweet and yellow	
Cherry, black	
Elm, rock	
Hickory, true and pecan	$P = 2040D^{3/2}$
Honey locust	
Locust, black	
Maple (hard), black and sugar	
Oak, commercial red and white	
Walnut, black	

the 8-in. spike will satisfy. A spike 8 in. long will be $\frac{3}{8}$ in. in diameter, and the safe load for one spike is determined from the formula $P = 1650D^{3/2}$.

$$P = 1650 \times (0.375)^{3/2} = 379 \text{ lb.}$$

$$\text{Number required} = \frac{3000}{379} = 8$$

No definite rules have been set up for the spacing of nails or spikes, but their spacing is largely dependent upon the resistance to splitting of the wood used. A general precaution to avoid splitting is to use a nail or spike with a diameter of not more than one-seventh the thickness of the lumber. Based on this rule, the maximum diameter of spike in the above problem should be $\frac{1}{7} \times 2.625 = 0.375$ in.

COMMON SCREWS

74. Withdrawal Resistance. As with nails, the withdrawal resistance of screws is related to the density of the piece and the diameter of the screw. The safe load becomes

$$P = 2040G^2D$$

where P = safe load in pounds per inch of total length.
G = specific gravity based on oven-dry weight and volume.
D = diameter of screw in inches.

The equation is based on screws inserted into the side grain of seasoned material and with a depth of penetration of not less than two-thirds the length of the screw. When screws are driven parallel to the grain or into the end of a piece use 75 per cent of the value obtained from the above equation. The equation values apply to the screw lengths and gages shown in Table 15.

Table 15. Screw Lengths and Gages Applicable in Formula for Withdrawal Resistance

Screw Length (Inches)	*Gage Limits*	*Screw Length (Inches)*	*Gage Limits*
$\frac{1}{2}$	1– 6	2	7–16
$\frac{3}{4}$	2–11	$2\frac{1}{2}$	9–18
1	3–12	3	12–20
$1\frac{1}{2}$	5–14		

When the wood is unseasoned use two-thirds of the values obtained from the equation, and when the wood is alternately wet and dry use three-fourths.

Table 16. Equations for Computing the Safe Lateral Loads for Wood Screws in Wood at 15 Per Cent Moisture Content Expressed in Pounds per Screw

[Wood Handbook, U. S. Dept. Agr., 1935]

Species of Wood	*Equation*
Softwoods:	
Cedar, northern and southern white	$P = 2520D^2$
Fir, balsam and commercial white	
Hemlock, eastern	
Pine, lodgepole, ponderosa, sugar, northern white, and western white	
Spruce, Engelmann, red, Sitka, and white	
Cedar, Alaska, incense, Port Orford, and western red	$P = 3240D^2$
Cedar, eastern red	
Cypress, southern	
Douglas fir (Rocky Mountain region)	
Hemlock, western	
Pine, Norway	
Redwood	
Tamarack	
Douglas fir (coast region)	$P = 3960D^2$
Larch, western	
Pine, southern yellow	
Hardwoods:	
Aspen and largetooth aspen	$P = 2520D^2$
Basswood	
Butternut	
Chestnut	
Cottonwood, black and eastern	
Poplar, yellow	
Alder, red	$P = 3480D^2$
Ash, black	
Birch, paper	
Elm, American and slippery	
Gum, black, red, and tupelo	
Hackberry	
Magnolia, cucumber	
Magnolia, evergreen	
Maple, bigleaf	
Maple (soft), red and silver	
Sugarberry	
Sycamore	
Ash, commercial white	$P = 4800D^2$
Ash, Oregon	
Beech	
Birch, sweet and yellow	
Cherry, black	
Elm, rock	
Hickory, true and pecan	
Honey locust	
Locust, black	$P = 4800D^2$
Maple (hard), black and sugar	
Oak, commercial red and white	
Walnut, black	

75. Lateral Resistance. The safe lateral load can be expressed by

$$P = KD^2$$

where P = safe lateral load in pounds per screw.
K = a constant varying with species.
D = diameter of shank in inches.

The equations for various species are given in Table 16.

The equations are based on a penetration into the member receiving the point of seven times the screw diameter. For a metal-to-timber joint, increase the safe loads 25 per cent.

For unseasoned lumber use two-thirds of the equation values, and when the lumber is alternately wet and dry use three-fourths.

When the load acts perpendicular to the grain and the screw is inserted in the end grain of the main member, the allowable lateral loads should be two-thirds of the equation values.

76. Prebored Holes. For withdrawal resistance the lead hole should have a diameter of about 90 per cent of the root diameter of the screw when hardwoods are used and about 70 per cent of the root diameter when softwoods are used.

For lateral resistance the part of the lead hole receiving the shank should be about the same diameter as the shank and that receiving the threaded portion should be about the same diameter as the root of the thread when hardwoods are used. For softwoods, the part of the lead hole receiving the shank should be about seven-eighths the diameter of the shank and that for the threaded portion should be about seven-eighths the diameter of the root of the thread.

77. Spacing. No definite rules for spacings and end and edge distances for wood screws have been determined, but the screws should be so placed as to prevent unusual splitting of the wood.

LAG SCREWS

78. General. Lag screws, which are essentially large screws requiring lead holes and a wrench to turn them into the wood, are used in place of bolts where it would be difficult to fasten a bolt or where the appearance of a nut on a surface would be objectionable.

Tests * on lag screws having a yield point of 45,000 lb. per sq. in. and an average ultimate tensile strength of 77,000 lb. per sq. in. indicate that the resistance to withdrawal depends upon the diameter of the lead hole, the diameter of the lag screw, the depth of penetration of the lag screw into the member receiving the point, the species, the specific gravity, the moisture content, and the direction of penetration of the lag screw with respect to the grain of the wood.

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* "Lag Screw Joints: Their Behavior and Design," by J. A. Newlin and J. M. Gahagan, *Tech. Bull.* 597, U. S. Dept. Agr., 1938.

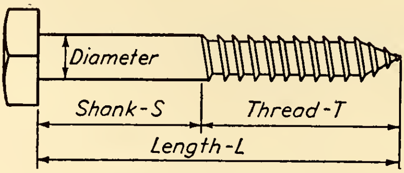

Fig. 32. Lag screw.

Withdrawal Resistance

79. Test Procedure.* The species used in testing lag screws in withdrawal were northern white pine, redwood, Douglas fir, southern pine, and white oak. A wide range in density of each species was used and the specimens were of sufficient width and length to permit the driving of screws of different diameters and lengths. The tests were made in a standard testing machine equipped with a special device for holding the screw head as illustrated in Fig. 33, and the speed of testing was 0.041 in. per minute.

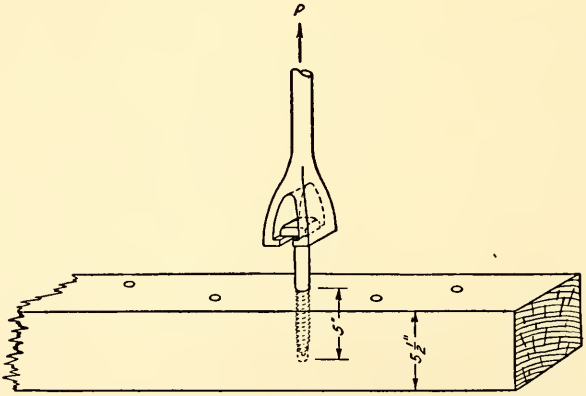

Fig. 33. Method of testing lag screws for direct withdrawal.

Three groups of direct-withdrawal tests were made (1) to determine the most efficient diameter of lead hole for each size of lag screw and for different species of wood, (2) to show the effects of different depths of driving, and (3) to show how differences in diameter of lag screw and in the quality of the wood affect the resistance to withdrawal from either side-grain or an end-grain surface of the specimen.

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* "Lag Screw Joints: Their Behavior and Design," by J. A. Newlin and J. M. Gahagan, *Tech. Bull.* 597, U. S. Dept. Agr., 1938.

80. Lead Holes. Results of tests on a ⅝-in. lag screw with various diameters of the lead hole and several species are illustrated in Chart 5.

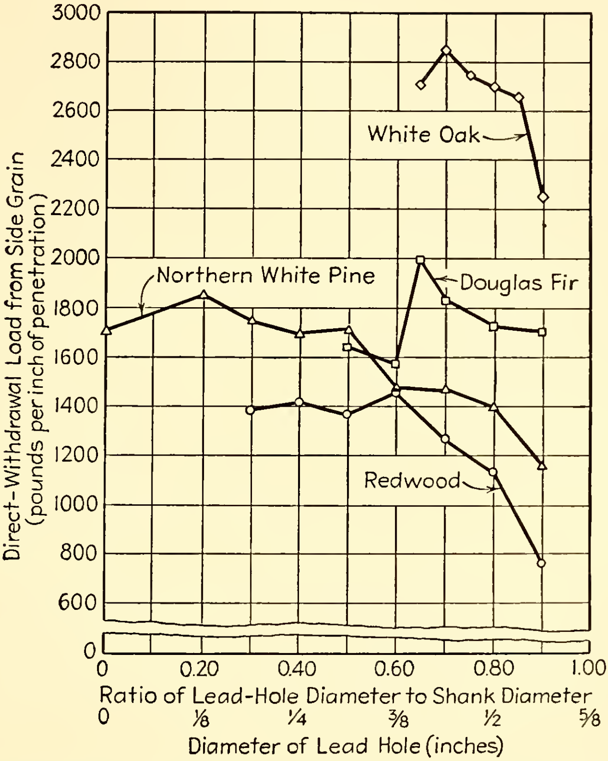

CHART 5. Relation of direct withdrawal of a ⅝-in. lag screw to diameter of lead hole, in air-dry wood.

Chart 5 indicates that the greatest withdrawal load is obtained when the ratio of the lead-hole diameter to the shank diameter for white oak is 0.70, for Douglas fir 0.65, for redwood 0.60, and for northern white pine a ratio of 0.50 or less.

Chart 6 illustrates the results of tests for various diameters of lag screws in lead holes of different diameters in air-dry redwood. There seems to be no consistent variation in load for different ratios of lead hole to shank diameter or for different sizes of lag screws for the same ratio of lead hole to shank diameter. For all practical purposes, however, the most effective range in diameter of lead hole in white oak is 65 to 85 per cent of the shank diameter, 60 to 75 per cent for southern pine and Douglas fir, and 40 to 70 per cent for redwood and northern white pine. The lead hole should have the same depth as the threaded portion of the lag screw. The smaller ratio applies to lag screws of small diameter and the larger ratio to large lag screws.

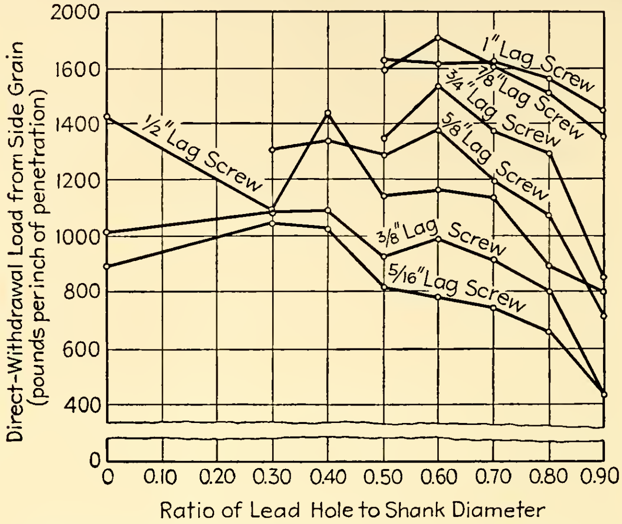

Chart 6. Resistance to direct withdrawal of lag screws of various diameters in lead holes of different diameters, in air-dry wood.

81. Depth of Penetration. Chart 7 represents the results of tests to determine the influence of depth of penetration of the threaded portion of the lag screw into the member receiving the point. As may be seen the resistance to withdrawal of lag screws of any diameter and any species of wood varies directly as the depth of penetration. For this reason all withdrawal test loads are reduced to pounds per inch of penetration.

82. Specific Gravity. No single property is as good a criterion of the resistance to direct withdrawal of a lag screw as the specific gravity of the wood. Chart 8 shows that the resistance to withdrawal of lag screws varies as the $\frac{3}{2}$ power of the specific gravity of the wood.

83. Diameter of Lag Screw. A very close relationship exists between the diameter of lag screws and their resistance to withdrawal. Chart 9 shows that for a given species the withdrawal loads per inch of penetration vary about as the $\frac{3}{4}$ power of the diameter of the lag screw.

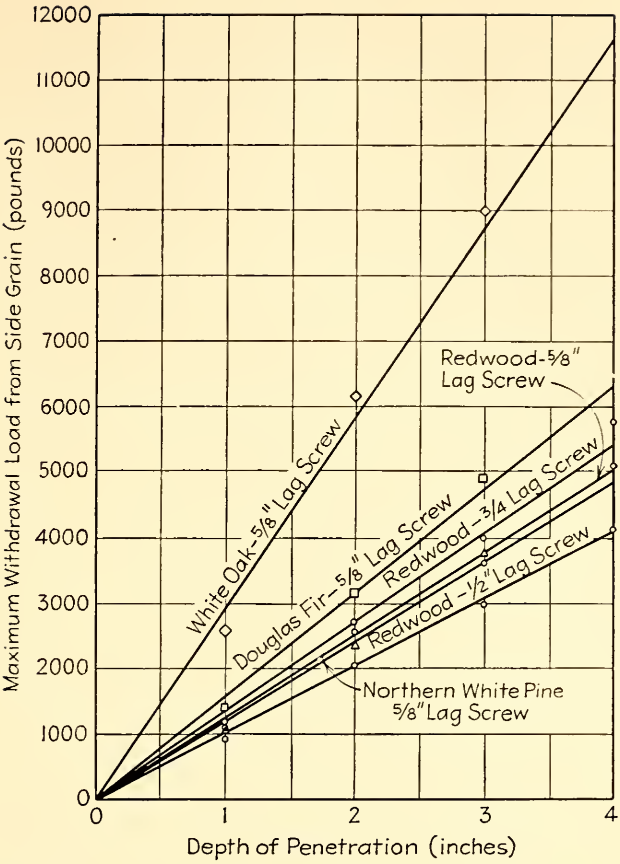

CHART 7. Resistance of lag screws to direct withdrawal for different depths of penetration using optimum diameter of lead hole.

84. Safe Withdrawal Loads. The safe withdrawal loads, with a safety factor of about 5, may be expressed in terms of a constant, the specific gravity of the wood, and the diameter of the lag screw such that

$$P = 1800G^{3/2}D^{3/4}$$

where P = allowable load per inch of penetration of the threaded portion of the lag screw in the side grain of seasoned wood.
G = specific gravity of oven-dry wood.
D = diameter of shank in inches.

The allowable load determined from the previous equation is based on seasoned lumber. For joints used under conditions where they will be alternately wet and dry use three-fourths of the equation values, and for joints that will be damp or wet most of the time use two-thirds.

For lag screws made of metal with an ultimate tensile strength other than 77,000 lb. per sq. in., the withdrawal loads should be adjusted in

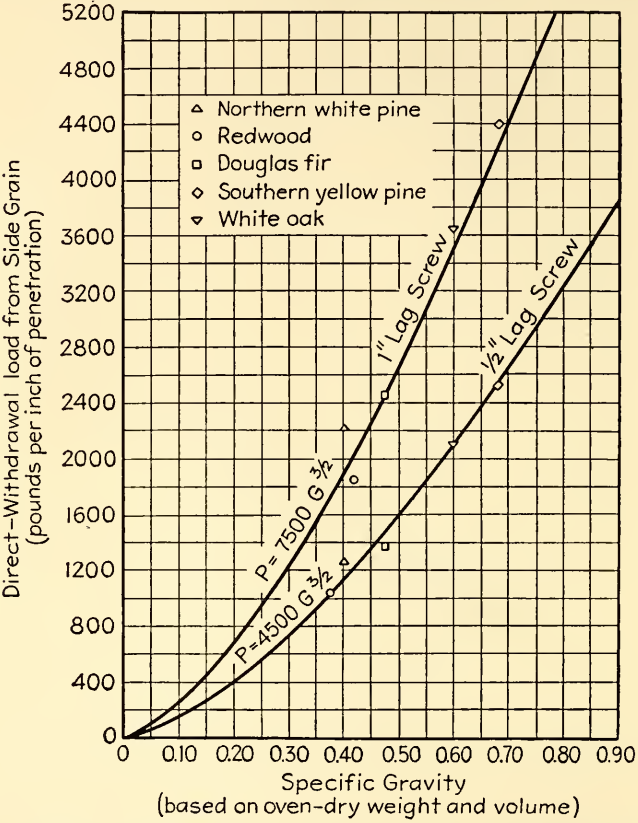

CHART 8. Relation between direct withdrawal loads of different-diameter lag screws and specific gravity of the wood.

proportion to the tensile strength. However, the allowable load should never exceed the allowable tensile strength of a lag screw based on the section at the root of the thread. In general, a penetration of the threaded portion equal to 7 times the shank diameter in the denser species and 10 to 12 times the shank diameter in the softer species will develop the ultimate tensile strength of the lag screw.

If the lag screw is inserted into the end of a piece use three-fourths of the value obtained from the previous equation.

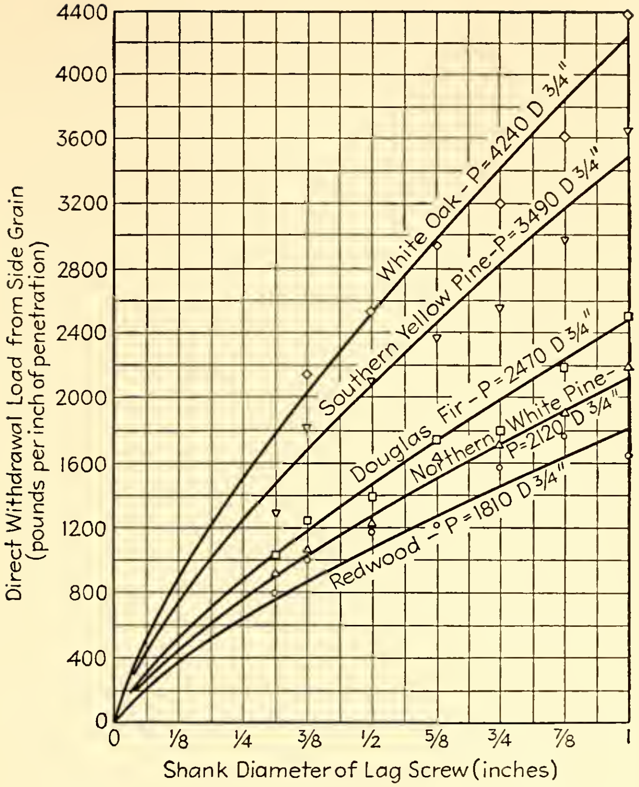

Chart 9. Resistance of lag screws of different diameters to direct withdrawal.

Lateral Resistance

85. Test Procedure. The same species as those used in the withdrawal tests were used in the tests to determine lateral resistance. The tests were made parallel to the grain only, and each test specimen consisted of two members, a cleat and a block of the same species, joined by a lag screw passing through the cleat and into the block. The position of the test specimen in the testing machine was such that the line of force passed downward approximately through the lag screw at the point where the block and cleat met. The lead hole in the cleat had the same diameter as the shank, and in the block it was the diameter which had previously been recommended for a particular species.

The testing machine had a speed of 0.015 in. per minute to a point just beyond the proportional limit and was then increased at intervals to 0.031 and 0.05 in. per minute until the test was completed.

86. Specific Gravity. The proportional limit loads for the different species were found to vary, regardless of the diameter of the lag screw, approximately as the ¾ power of the specific gravity. The results of these tests are shown in Chart 10.

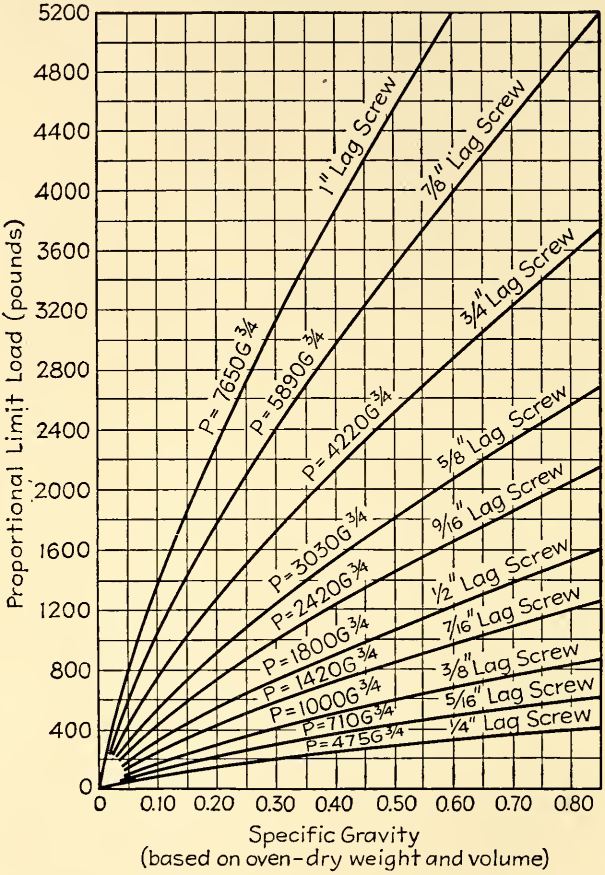

CHART 10. Relation between resistance to lateral displacement of lag screws of different diameters and the specific gravity of the wood. Screws were in side grain.

86. Diameter of Lag Screw. The proportional limit loads were found to vary as the square of the diameter of the lag screw, as illustrated in Chart 11.

87. Thickness of Cleat. A decided increase in the load occurred with an increase in the ratio of cleat thickness to the shank diameter of the lag screw up to a ratio of about 7 to 1. Beyond this ratio little increase in the proportional limit loads was apparent, and below a 3.5 to 1 ratio the loads were somewhat irregular.

The results of the tests on a basis of the percentage change in load with change in the cleat thickness ratio are illustrated in Chart 12.

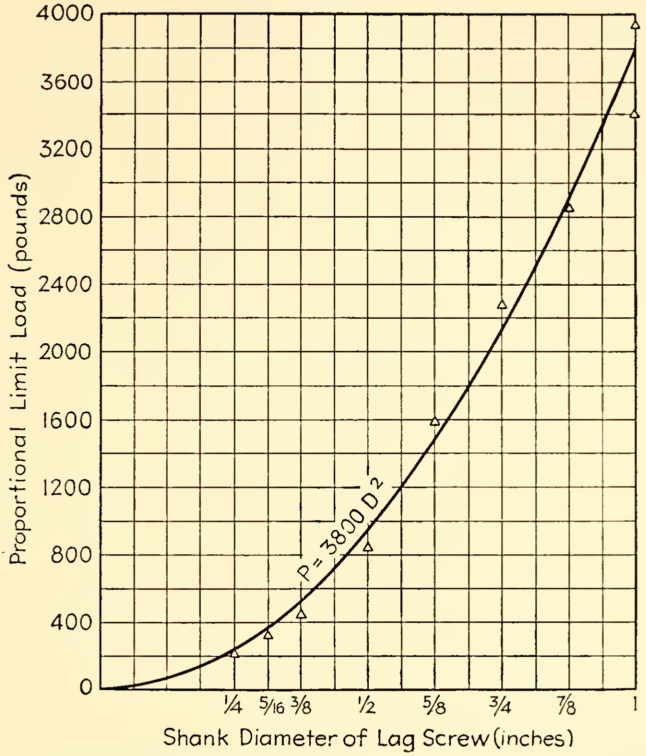

Chart 11. Relation between the diameter of lag screws and the resistance to lateral displacement in northern white pine. Screws were in side grain.

88. Depth of Lag Screw in the Block. If a constant ratio of cleat thickness of about 3.5 is used with lag screws of different diameters, a depth of penetration in the main member receiving the point of about 7 times the diameter of the lag screw for the harder species and about 11 to 12 times for the softer species will develop the full strength of the joint. Higher cleat thickness ratios require greater depths of penetration; for smaller ratios the cleat thickness may be reduced.

89. Length of Shank in Block. In Chart 13 the effect of the ratio of the depth of the shank in the block receiving the point to the shank diameter is plotted against the percentage of the load. The point representing the load at zero penetration of the shank into the block is taken

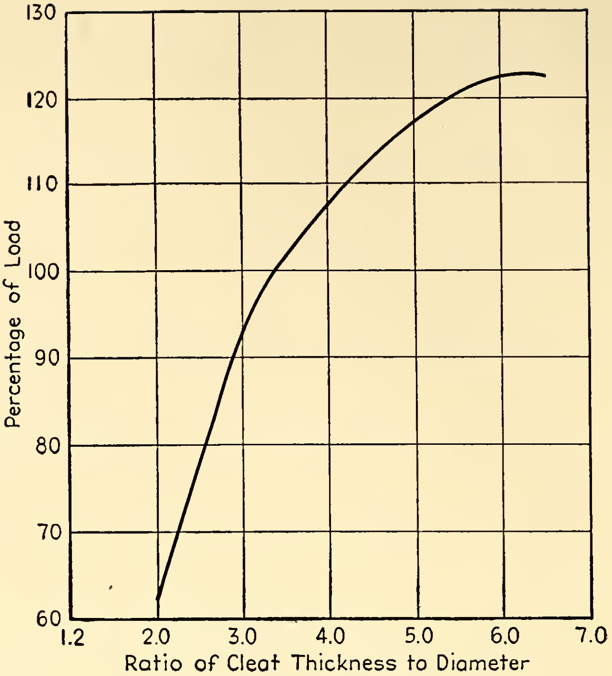

Chart 12. Influence on load of the ratio of cleat thickness to the diameter of the shank of a lag screw.

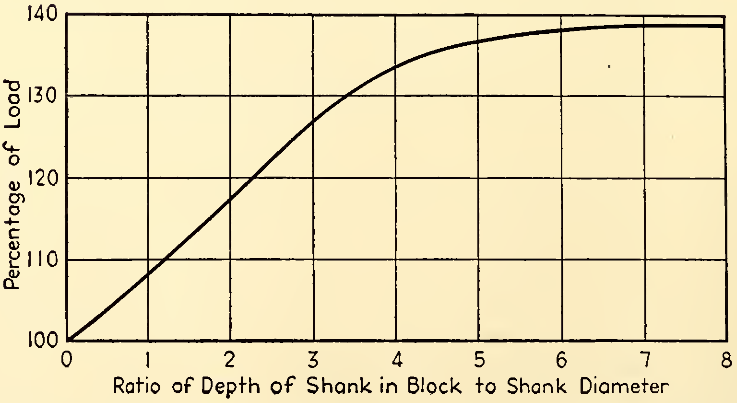

Chart 13. Influence on load of the ratio of the length of shank in block to the shank diameter.

as 100 per cent. As the ratio increases, up to a point where the ratio is about 7 to 1, the load increases.

90. Safe Lateral Loads. The allowable lateral loads parallel to the grain for lag screws inserted into the side grain of seasoned wood may be computed from the equations in Table 17 but must be modified in accordance with the accompanying text.

Table 17. Equations for Computing Safe Lateral Resistance of Lag Screws

[*Tech. Bull.* 597, U. S. Dept. Agr.]

Group	Species of Wood	Equation	Group	Species of Wood	Equation
1	Cedar, northern and southern white Fir, balsam and commercial white Hemlock, eastern Pine, ponderosa, sugar, northern white, and western white Spruce, Engelmann, red, Sitka, and white	$P_1 = 1800D^2$	3	Ash, black Birch, paper Douglas fir (coast type) Elm (soft), American and (gray) slippery Gum, black, red, and tupelo Larch, western Maple (soft), red and silver Pine, southern yellow Sycamore	$P_1 = 2280D^2$
2	Aspen and largetooth aspen Basswood Cedar, Alaska, Port Orford, and western red Chestnut Cottonwood, black and eastern Cypress, southern Douglas fir (Rocky Mountain type) Hemlock, western Pine, Norway Redwood Tamarack Yellow poplar	$P_1 = 2040D^2$	4	Ash, commercial white Beech Birch, sweet and yellow Elm, rock Hickory, true and pecan Maple (hard), black and sugar Oak, commercial red and white	$P_1 = 2640D^2$

The equations for safe lateral resistance are based on a cleat (wood side piece) thickness of 3.5 times the diameter of the shank and a penetration of the unthreaded shank to the plane of contact between the side and main member. For other conditions, the safe lateral loads obtained from the equations in Table 17 should be multiplied by the percentages taken from Charts 12 and 13.

When the unthreaded portion of the shank does not extend to the plane of contact between the side members, the allowable load should be determined on the basis of the actual penetration of the shank in the side member. The minimum value should be taken as 80 per cent if it is assumed that the thread extends to the head of the screw or the face

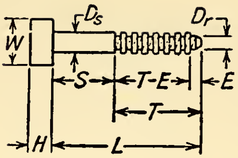

Table 18. Dimensions of Standard Lag Screws

D = nominal diameter.
Ds = *D* = diameter of shank.
Dr = diameter at root of thread.
W = width of bolt head across flats.
N = number of threads per inch.
H = height of bolt head.
L = nominal length of bolt.
S = length of shank.
T = length of thread.
E = length of tapered tip.

Nominal Length of Bolt *L* in Inches[1]	Item	Dimensions of Lag Bolt with Nominal Diameter *D* of									
		¼	5/16	⅜	7/16	½	9/16	⅝	¾	⅞	1
All lengths	*Ds* = *D*	0.250	0.3125	0.375	0.4375	0.500	0.5625	0.625	0.750	0.875	1.000
	Dr	0.173	0.227	0.265	0.328	0.371	0.435	0.471	0.579	0.683	0.780
	E	3/16	¼	¼	9/32	5/16	⅜	⅜	7/16	½	9/16
	H	11/64	13/64	¼	19/64	21/64	⅜	27/64	½	19/32	21/32
	W	⅜	½	9/16	⅝	¾	⅞	15/16	1⅛	15/16	1½
	N	10	9	7	7	6	6	5	4½	4	3½
1	*S*	¼	¼	¼	¼	¼	....	....	....	....	....
	T	¾	¾	¾	¾	¾	....	....	....	....	....
	T–E	9/16	½	½	15/32	7/16	....	....	....	....	....
1½	*S*	⅜	⅜	⅜	⅜	⅜	....	....	....	....	....
	T	1⅛	1⅛	1⅛	1⅛	1⅛	....	....	....	....	....
	T–E	15/16	⅞	⅞	27/32	13/16	....	....	....	....	....
2	*S*	½	½	½	½	½	½	½	....	....	....
	T	1½	1½	1½	1½	1½	1½	1½	....	....	....
	T–E	1 5/16	1¼	1¼	1 7/32	1 3/16	1⅛	1⅛	....	....	....
2½	*S*	1	⅞	⅞	¾	¾	¾	¾	....	....	....
	T	1½	1⅝	1⅝	1¾	1¾	1¾	1¾	....	....	....
	T–E	1 5/16	1⅜	1⅜	1 15/32	1 7/16	1⅜	1⅜	....	....	....
3	*S*	1	1	1	1	1	1	1	1	1	1
	T	2	2	2	2	2	2	2	2	2	2
	T–E	1 13/16	1¾	1¾	1 23/32	1 11/16	1⅝	1⅝	1 9/16	1½	1 7/16
4	*S*	1½	1½	1½	1½	1½	1½	1½	1½	1½	1½
	T	2½	2½	2½	2½	2½	2½	2½	2½	2½	2½
	T–E	2 5/16	2¼	2¼	2 7/32	2 3/16	2⅛	2⅛	2 1/16	2	1 15/16
5	*S*	2	2	2	2	2	2	2	2	2	2
	T	3	3	3	3	3	3	3	3	3	3
	T–E	2 13/16	2¾	2¾	2 23/32	2 11/16	2⅝	2⅝	2 9/16	2½	2 7/16
6	*S*	2½	2½	2½	2½	2½	2½	2½	2½	2½	2½
	T	3½	3½	3½	3½	3½	3½	3½	3½	3½	3½
	T–E	3 5/16	3¼	3¼	3 7/32	3 3/16	3⅛	3⅛	3 1/16	3	2 15/16
7	*S*	3	3	3	3	3	3	3	3	3	3
	T	4	4	4	4	4	4	4	4	4	4
	T–E	3 13/16	3¾	3¾	3 23/32	3 11/16	3⅜	3⅝	3 9/16	3½	3 7/16
8	*S*	3½	3½	3½	3½	3½	3½	3½	3½	3½	3½
	T	4½	4½	4½	4½	4½	4½	4½	4½	4½	4½
	T–E	4 5/16	4¼	4¼	4 7/32	4 3/16	4⅛	4⅛	4 1/16	4	3 15/16
9	*S*	4	4	4	4	4	4	4	4	4	4
	T	5	5	5	5	5	5	5	5	5	5
	T–E	4 13/16	4¾	4¾	4 23/32	4 11/16	4⅝	4⅝	4 9/16	4½	4 7/16
10	*S*	4¾	4¾	4¾	4¾	4¾	4¾	4¾	4¾	4¾	4¾
	T	5¼	5¼	5¼	5¼	5¼	5¼	5¼	5¼	5¼	5¼
	T–E	5 1/16	5	5	4 31/32	4 15/16	4⅞	4⅞	4 13/16	4¾	4 11/16
11	*S*	5½	5½	5½	5½	5½	5½	5½	5½	5½	5½
	T	5½	5½	5½	5½	5½	5½	5½	5½	5½	5½
	T–E	5 9/32	5¼	5¼	5 7/32	5 3/16	5⅛	5⅛	5 1/16	5	4 15/16
12	*S*	6	6	6	6	6	6	6	6	6	6
	T	6	6	6	6	6	6	6	6	6	6
	T–E	5 13/16	5¾	5¾	5 23/32	5 11/16	5⅝	5⅝	5 9/16	5½	5 7/16

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[1] Length of thread *T* on intervening bolt lengths is the same as that of the next shorter bolt length listed. The length of thread *T* on standard bolt lengths L in excess of 12 in. is equal to one-half the bolt length ($L/2$).

of the washer, and the percentage to be used as a multiplier of the load obtained from the equations in Table 17 may be found as follows:

$$100 - 20\left(\frac{t_1 + t_2 - L_1}{t_2}\right)$$

where t_1 = thickness of washer.
t_2 = thickness of side member or cleat.
L_1 = actual length of unthreaded shank.

The equations in Table 17 are based on a minimum penetration of the threaded portion of the lag screw into the main member of 11 times the shank diameter for Group 1 (softer woods) to 7 times for Group 4 (harder woods). When the ratio of the thickness of the side member to the shank diameter is greater than 3.5, and the ratio of the penetration of the shank into the main member to the shank diameter is greater than zero, greater depths of penetration of the threaded portion in the main member are required to develop the percentages given in Charts 12 and 13. The required penetration of the threaded portion into the main member should be obtained by multiplying the standard penetration by the square root of the product of the two percentages divided by 100. For example, if the ratio of thickness of side member to diameter of shank is 4.0 the percentage of load from Chart 12 is 107 per cent, and if the ratio of penetration of shank in main member to shank diameter is 1.5 the percentage load is 112.5 per cent. The square root of the product of these two percentages divided by 100 is 1.1. If the normal penetration of the threaded portion of the shank should be 9 times the diameter, the required penetration due to the above percentages is equal to 1.1 × 9 or 9.9. If this required penetration cannot be obtained, the allowable loads should be reduced in direct proportion of the actual penetration to that required, but no increase should be taken if the penetration is greater than that required.

To clarify the foregoing paragraph Table 19 may be used as a guide.

When the lag screw is inserted into the side grain of wood but the load is applied perpendicular to the grain, the allowable loads may be determined by multiplying the loads obtained from the equations in Table 17 by the factors in Table 20.

The Group numbers in Table 20 refer to the species designated by each number in Table 21.

When the load acts at an angle to the grain, the allowable load may be computed by the formula given in Chapter III.

When the lag screw is inserted into the end grain of wood and the load acts perpendicular to the grain, the allowable lateral loads should

Table 19. Examples of Percentages as Multipliers of Allowable Lateral Loads

t_2 = thickness of side member.
D = diameter of shank.
L = length of shank in main member.
L_2 = penetration of threaded portion in block; assume required value equal to $9D$, which would be true for Douglas fir.

	t_2/D	L/D	*Required* L_2/D	*Actual* L_2/D	*Percentage to Use as Multiplier*
	3.5	0	$100 \times 9 = 9.0$	9.0	
Per cent	100	100		100	100
	3.5	0	$100 \times 9 = 9.0$	7.2	
Per cent	100	100		80	80
	2.5	1.5	$87 \times 9 = 7.8$	7.8	
Per cent	77.5	$112.5 = 87$	$\times$	100	87
	4.0	1.5	$\sqrt{120} \times 9 = 9.9$	9.0	
Per cent	107	$112.5 = 120$	$\times$	90	108
	4.0	$\left(\frac{t_1 + t_2 - L_1}{t_2}\right) = 0.3$	$\sqrt{101} \times 9 = 9.1$	9.5	
Per cent	107	$94 = 101$		100	101

Table 20. Factors for Computing Allowable Lateral Loads When the Load Acts Perpendicular to the Grain and the Lag Screw Is Inserted into the Side Grain

Lag Screw Diameter	Factor				
	Group 1	Group 2	Group 3	Group 4	Group 5
$\frac{3}{16}$	1.00	1.00	1.00	1.00	1.00
$\frac{1}{4}$	0.94	1.00	1.00	0.89	1.00
$\frac{5}{16}$	0.82	0.87	0.89	0.77	0.91
$\frac{3}{8}$	0.73	0.78	0.80	0.69	0.82
$\frac{7}{16}$	0.67	0.72	0.74	0.64	0.76
$\frac{1}{2}$	0.62	0.67	0.68	0.59	0.70
$\frac{5}{8}$	0.57	0.61	0.62	0.54	0.64
$\frac{3}{4}$	0.53	0.56	0.57	0.50	0.59
$\frac{7}{8}$	0.50	0.53	0.54	0.47	0.56
1	0.48	0.51	0.52	0.45	0.53

Table 21. Grouping of Species for Use with Table 20

Group	Species of Wood
1	Ash, black Aspen and largetooth aspen Basswood Birch, paper Butternut Cedar, northern and southern Chestnut Cottonwood, black and eastern Fir, balsam and commercial white Hemlock, eastern Pine, lodgepole, ponderosa, sugar, northern white, and western white Poplar, yellow Spruce, Engelmann, red, Sitka, and white
2	Cedar, Alaska, incense, Port Orford, and western red Douglas fir (Rocky Mountain region) Hemlock, western Pine, Norway
3	Cedar, eastern red Cypress, southern Douglas fir (coast region) Larch, western Pine, southern yellow Redwood Tamarack
4	Alder, red Elm, American and slippery Gum, black, red, and tupelo Hackberry Magnolia, cucumber and evergreen Maple, bigleaf and red and silver (soft) Sugarberry Sycamore
5	Ash, commercial white and Oregon Beech Birch, sweet and yellow Cherry, black Elm, rock Hickory, true and pecan Locust, black Locust, honey Maple (hard), black and sugar Oak, commercial red and white Walnut, black

be taken as two-thirds of the values for loads acting perpendicular to the grain with the lag screw in the side grain.

91. Steel Splice Plates. When steel side pieces are used instead of wood, the allowable lateral loads given by formulas for parallel to grain loading should be increased by 25 per cent. No increase should be taken for the use of metal splice plates when the load acts perpendicular or at an angle to the grain.

92. Spacing. The spacings, end and edge distances and net section, should be the same as for joints with bolts of a diameter equal to the shank diameter of the lag screw. See the section entitled "Bolts."

DRIFT BOLTS

93. General. A drift bolt is a round or square steel or iron pin without threads. Drift bolts are usually driven into holes about 1/8 in. smaller than their diameter. The only test results available are those with respect to withdrawal resistance, and these have not been extensive enough to suggest safe working loads.

94. Withdrawal Resistance. The resistance to withdrawal of round drift bolts driven perpendicular to the grain into seasoned wood may be expressed by the formula

$$P = 6000G^2D$$

where P = maximum withdrawal load per lineal inch of penetration.
G = specific gravity based on weight and volume when oven-dry.
D = diameter of drift bolt in inches.

Some species will fall above the equation value and others below it. For southern pine 80 per cent of the equation value should be used; and a factor of safety of 5 being introduced. the safe working load may be expressed as $P_1 = 960G^2D$.

BOLTS

95. General. Tests * to determine the strength of wood joints fastened with common bolts having a yield point of 45,000 lb. per sq. in. indicated that there was a definite relationship between the length and the diameter of the bolt and the proportional limit of the average bearing value of the wood. This means that the average bearing value of the wood may be expressed in terms of the ratio of the length of the bolt to its diameter in the main member of a joint. Furthermore, the tests showed that this relationship exists regardless of the size of the bolt; that is, a ½-in. bolt in timber 4 in. thick will have the same average bearing value as a 1-in. bolt in a timber 8-in. thick. This is true for loads parallel to the grain; however, when the load is applied perpendicular to the grain, it is necessary, within certain limits, to apply a corrective factor depending upon the size of the bolt.

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* "The Bearing Strength of Wood Under Bolts," by G. W. Trayer, *Tech. Bull.* 332, U. S. Dept. Agr., 1932.

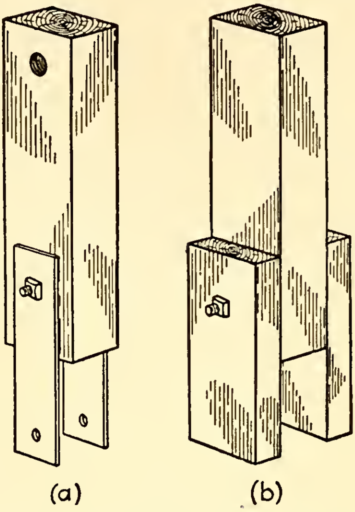

Fig. 34. Bolted test specimens to which load was applied parallel to grain (*a*) with metal splice plates, and (*b*) with wood splice plates.

96. Testing Procedure. The test specimens are illustrated in Figs. 34 and 35, and in all tests to determine the bearing strength of wood under bolts of various diameters and lengths only one bolt was used in each specimen. In a series of tests to determine the effect of shrinkage subsequent to assembly four bolts were used, and in tests to determine end and edge distances and spacings the number of bolts in each specimen varied from one to eight.

Gages were attached at each end of the bolt to measure the slip in the joint, and the load was applied at the rate of 0.026 in. per minute.

The bolt diameters ranged from ¼ to 1 in., and the thickness of the main members ranged from 2 to 12 times the bolt diameter.

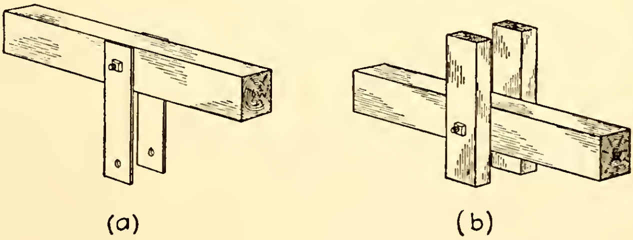

Fig. 35. Bolted test specimen to which load was applied perpendicular to grain (*a*) with metal splice plates, and (*b*) with wood splice plates.

The species of wood used included Douglas fir, southern pine, Sitka spruce, oak, and maple. The test specimens for all tests except those to determine the effect of shrinkage were cut from seasoned material. The properties of the test specimens were determined by testing small control specimens cut from each piece.

97. Proportional Limit. The proportional limit stress is taken as the average stress under the bolt when the slip in the joint ceases to be proportional to the load. This is not a true proportional limit because there will be a slight set in the joint when the load is removed. This set is due to the embedding of the bolt in the wood fibers. However, the set is very small, being only a few hundredths of an inch, and repeated load tests show that the joint can sustain the proportional limit load repeatedly without injury or further set.

Chart 14 is a typical curve showing the relation between load on the bolt and slip in the joint. The curve represents the results of a joint made with metal splice plates connected to a 2-in. timber by means of one ½-in. bolt.

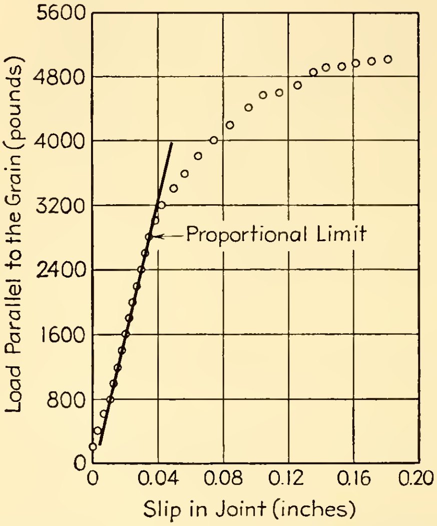

CHART 14. Relation between load parallel to the grain and slip in the joint for an individual test specimen.

98. Basic Stresses. From the tests performed at the Forest Products Laboratory, basic stresses have been developed for calculating the safe loads for bolted joints. These stresses are for use in the design of bolted joints only and are not to be confused with the working stresses for structural lumber.

The basic stresses apply irrespective of the grade of lumber used and are for seasoned timbers used in a dry inside location.

Table 22. Basic Stresses for Calculating Safe Loads for Bolted Joints

Group	*Species of Wood*	*Basic Stress (Pounds per Square Inch)* Parallel with the Grain	*Basic Stress (Pounds per Square Inch)* Perpendicular to the Grain
Softwoods (conifers):			
1	Cedar, northern and southern; fir, balsam and commercial white; hemlock, eastern; pine, lodgepole, ponderosa, sugar, northern white, and western white; spruce, Engelmann, red, Sitka, and white	960	180
2	Cedar, Alaska, incense, Port Orford, and western red; Douglas fir (Rocky Mountain); hemlock, western; pine, Norway	1200	240
3	Cedar, eastern red; cypress, southern; Douglas fir (coast region); larch, western; pine, southern yellow; redwood; tamarack	1560	330
Hardwoods (broad-leaved species):			
4	Ash, black; aspen and largetooth aspen; basswood; birch, paper; butternut; chestnut; cottonwood, black and eastern; poplar, yellow	1110	210
5	Alder, red; elm, American and slippery; gum, black, red, and tupelo; hackberry; magnolia, cucumber and evergreen; maple, bigleaf and red and silver (soft); sugarberry; sycamore	1440	300
6	Ash, commercial white and Oregon; beech; birch, sweet and yellow; cherry, black; elm, rock; hickory, true and pecan; honey locust; locust, black; maple (hard), black and sugar; oak, commercial red and white; walnut, black	1800	480

99. Varying L/D Ratio. Charts 15 and 16 show the working stress to use in calculating the safe load of a bolted joint for different L/D ratios. The charts are based on a three-member joint (double shear),

Table 23. Diameter Factors for Loads Perpendicular to the Grain

Diameter of Bolt (Inches)	*Diameter Factor*	*Diameter of Bolt (Inches)*	*Diameter Factor*
$\frac{1}{4}$	2.50	$1\frac{1}{4}$	1.19
$\frac{3}{8}$	1.95	$1\frac{1}{2}$	1.14
$\frac{1}{2}$	1.68	$1\frac{3}{4}$	1.10
$\frac{5}{8}$	1.52	2	1.07
$\frac{3}{4}$	1.41	$2\frac{1}{2}$	1.03
$\frac{7}{8}$	1.33	3 and over	1
1	1.27		

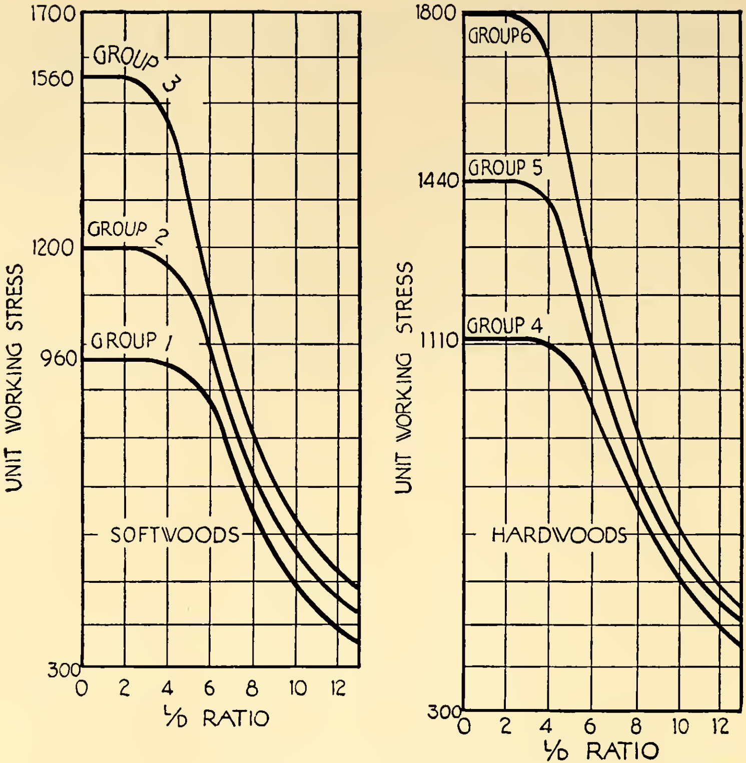

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CHART 15. Safe working stresses for bolted joints for any L/D ratio. Load applied parallel to grain of main member. Values based on use of metal splice plates. For wood splice plates use 80 per cent of these values. Common bolts—yield point 45,000 lb. per sq. in.

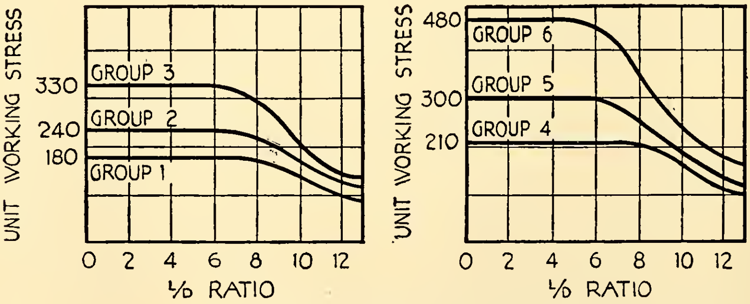

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CHART 16. Safe working stresses for bolted joints for any L/D ratio. Load applied perpendicular to grain of main member. Values are based on use of metal or wood splice plates and common bolts. Yield point—45,000 lb. per sq. in.

and the length of the bolt is measured as the thickness of the main member. When wood splice plates are used they are to be one-half the thickness of the main member.

Table 23 lists the diameter factors of different bolts to be used as a multiplier in determining the safe load when the load is applied perpendicular to the grain.

100. Illustrative Problems. (*a*) Consider a tension joint as shown in Fig. 36 with metal and wood splice plates. The main member is 6 in. thick (5½ in. actually) and consists of two pieces of seasoned southern pine joined by the splice plates and eight common bolts ⅞ in. in diameter. The joint is to be used in a dry inside location. The L/D ratio is $5.5/0.875 = 6.28$, and for this ratio Chart 15 gives a safe stress with metal splice plates of 1075 lb. per sq. in. With wood splice plates the safe stress is $1075 \times 0.8 = 860$ lb. per sq. in.

Safe load for one bolt $= 1075 \times 5.5 \times 0.875 = 5180$ lb.
Safe load for four bolts $= 5180 \times 4 = 20{,}700$ lb.

With wood splice plates each one-half the thickness of the main member the

Safe load for one bolt $= 5180 \times 0.8 = 4140$ lb.
Safe load for four bolts $= 4140 \times 4 = 16{,}580$ lb.

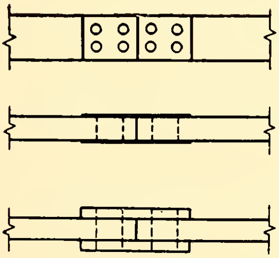

FIG. 36. Bolted joint with load parallel to grain.

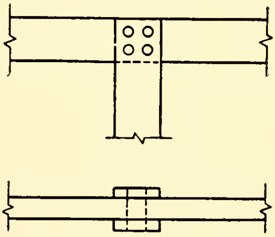

FIG. 37. Bolted joint with load perpendicular to grain.

(*b*) The joint shown in Fig. 37 consists of a main member 6 in. thick with splice plates 3 in. thick and is seasoned southern pine to be used in a dry inside location. Four common bolts ⅞ in. in diameter are used to make the joint. The basic stress is 330 lb. per sq. in., and the L/D ratio is $5.5/0.875 = 6.28$. For this ratio Chart 16 gives a safe stress of 329 lb. per sq. in. The diameter factor is 1.33.

Safe load for one bolt $= 329 \times 1.33 \times 5.5 \times 0.875 = 2100$ lb.
Safe load for four bolts $= 2100 \times 4 = 8400$ lb.

Table 24. Standard Bolt Sizes

[A.I.S.C. Manual of Steel Construction]

Dia. of Bolt (in.)	Series	Head					Dia. of Bolt (in.)	Series	Nut				
		Hexagon		Height (in.)	Square				Hexagon		Height (in.)	Square	
		Diameter (in.)			Diameter (in.)				Diameter (in.)			Diameter (in.)	
		Long	Short		Long	Short			Long	Short		Long	Short
1/4	American Standard Regular	7/16	3/8	3/16	1/2	3/8	1/4	American Standard Regular	1/2	7/16	1/4	5/8	7/16
3/8		5/8	9/16	1/4	3/4	9/16	3/8		11/16	5/8	5/16	7/8	5/8
1/2		7/8	3/4	5/16	1	3/4	1/2		15/16	13/16	7/16	1 1/8	13/16
5/8		1 1/16	15/16	7/16	1 5/16	15/16	5/8		1 1/8	1	9/16	1 3/8	1
3/4		1 5/16	1 1/8	1/2	1 9/16	1 1/8	3/4		1 5/16	1 1/8	11/16	1 9/16	1 1/8
7/8		1 1/2	1 5/16	9/16	1 13/16	1 5/16	7/8		1 1/2	1 5/16	3/4	1 13/16	1 5/16
1		1 11/16	1 1/2	5/8	2 1/16	1 1/2	1		1 11/16	1 1/2	7/8	2 1/16	1 1/2
1 1/8		1 15/16	1 11/16	3/4	2 5/16	1 11/16	1 1/8		1 15/16	1 11/16	1	2 5/16	1 11/16
1 1/4		2 1/8	1 7/8	13/16	2 9/16	1 7/8	1 1/4		2 1/8	1 7/8	1 1/8	2 9/16	1 7/8
1 3/8		2 3/8	2 1/16	15/16	2 13/16	2 1/16	1 3/8		2 3/8	2 1/16	1 1/4	2 13/16	2 1/16
1 1/2		2 9/16	2 1/4	1	3 1/16	2 1/4	1 1/2		2 9/16	2 1/4	1 5/16	3 1/8	2 1/4
1 3/8	American Standard Regular	2 3/8	2 1/16	15/16	2 13/16	2 1/16	1 3/8	American Standard Heavy	2 1/2	2 3/16	1 3/8	3	2 3/16
1 1/2		2 9/16	2 1/4	1	3 1/16	2 1/4	1 1/2		2 11/16	2 3/8	1 1/2	3 1/4	2 3/8
1 5/8		2 3/4	2 7/16	1 1/16	3 3/8	2 7/16	1 5/8		2 15/16	2 9/16	1 5/8	3 1/2	2 9/16
1 3/4		3	2 5/8	1 3/16	3 5/8	2 5/8	1 3/4		3 1/8	2 3/4	1 3/4	3 3/4	2 3/4
1 7/8		3 3/16	2 13/16	1 1/4	3 7/8	2 13/16	1 7/8		3 3/8	2 15/16	1 7/8	4 1/16	2 15/16
2		3 7/16	3	1 5/16	4 1/8	3	2		3 9/16	3 1/8	2	4 5/16	3 1/8
2 1/4		3 7/8	3 3/8	1 1/2	4 5/8	3 3/8	2 1/4		4	3 1/2	2 1/4	4 13/16	3 1/2
2 1/2		4 1/4	3 3/4	1 11/16	5 1/8	3 3/4	2 1/2		4 7/16	3 7/8	2 1/2	5 5/16	3 7/8
2 3/4		4 11/16	4 1/8	1 13/16	5 11/16	4 1/8	2 3/4		4 7/8	4 1/4	2 3/4	5 13/16	4 1/4
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4		6 7/8	6	2 11/16	8 1/4	6	4		7	6 1/8	4	8 7/16	6 1/8
4 1/4		7 1/4	6 3/8	2 13/16	8 3/4	6 3/8	4 1/4		7 7/16	6 1/2	4 1/4	8 15/16	6 1/2
4 1/2		7 11/16	6 3/4	3	9 1/4	6 3/4	4 1/2		7 13/16	6 7/8	4 1/2	9 7/16	6 7/8
4 3/4		8 1/8	7 1/8	3 3/16	9 13/16	7 1/8	4 3/4		8 1/4	7 1/4	4 3/4	9 15/16	7 1/4
5		8 9/16	7 1/2	3 5/16	10 5/16	7 1/2	5		8 11/16	7 5/8	5	10 1/2	7 5/8
5 1/4		9	7 7/8	3 1/2	10 13/16	7 7/8	5 1/4		9 1/8	8	5 1/4	11	8
5 1/2		9 3/8	8 1/4	3 11/16	11 5/16	8 1/4	5 1/2		9 9/16	8 3/8	5 1/2	11 1/2	8 3/8
5 3/4		9 13/16	8 5/8	3 13/16	11 13/16	8 5/8	5 3/4		10	8 3/4	5 3/4	12	8 3/4
6		10 1/4	9	4	12 3/8	9	6		10 3/8	9 1/8	6	12 1/2	9 1/8

(*c*) In calculating the bearing stress at an angle to the grain the Hankinson formula, given in Article 55, is recommended. The joint shown in Fig. 38 will serve as an example to illustrate the method used in calculating the safe load for a joint when the load is applied at an angle to the grain.

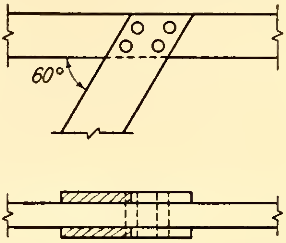

FIG. 38. Bolted joint with load at an angle to grain.

The center member is 6 in. thick and the side members are 3 in. thick; the joint consists of seasoned southern pine to be used in a dry inside location. Four common bolts ⅞ in. in diameter are used to make the joint. The joint is the same size as that used in the previous examples in which the safe stress parallel to the grain was 800 lb. per sq. in., and the safe stress perpendicular to the grain was 329 times 1.33, or 437 lb. per sq. in.

The sine^2 of 60° is 0.75 and the cosine^2 of 60° is 0.25. The safe stress when the angle between the direction of load and the direction of grain is 60° then becomes

$$N = \frac{800 \times 437}{800 \times 0.75 + 437 \times 0.25} = 493 \text{ lb. per sq. in.}$$

$$\text{Safe load for four bolts} = 493 \times 4 \times 5.5 \times 0.875 = 9480 \text{ lb.}$$

101. Design Details. When joints are exposed to the weather use three-fourths of the values obtained from the procedure outlined in the previous article, and if wet or damp most of the time use two-thirds. For green lumber, which is allowed to season under load, use 33⅓ per cent of the calculated safe load for dry lumber.

If the side members in a three-member joint are thicker than one-half the thickness of the main member, the safe load for the joint is determined in the same manner as for a joint with side members one-half the thickness of the main member. However, if the side members are less than one-half the thickness of the main member, consider the length of the bolt as twice the thickness of the thinnest side member used. For example, for 3-in. side members and an 8-in. main member, use 6 in.

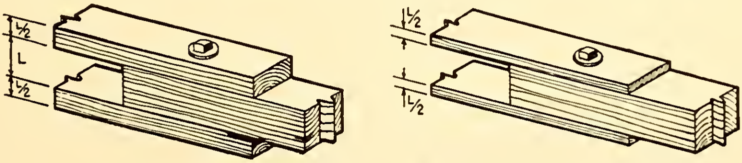

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FIG. 39. Values of L for L/D ratios in three-member joints.

When a joint consists of two members of equal thickness, calculate the safe load by considering a main member twice as thick as one of the members and reducing the result by 50 per cent. If the members in a two-member joint are of unequal thickness, calculate the safe load based on a piece twice the thickness of the thinnest member, and reduce this safe load by 50 per cent. If a joint consists of four members of equal thickness, the safe load to be used is one and one-half times the calculated safe load for a piece the thickness of one of the members.

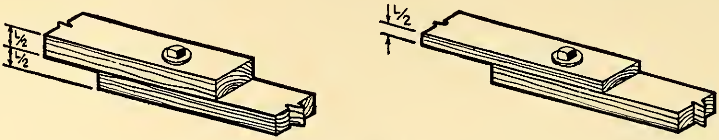

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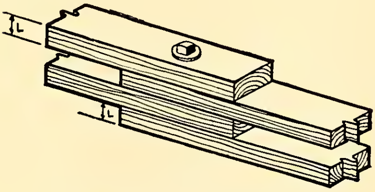
Fig. 40. Values of L for L/D ratios in two-member joints.

Figures 42 and 43 illustrate the method of measuring the spacings and the end and edge distances.

The maximum efficiency of a joint when the load is parallel to the grain is obtained when the L/D ratio is 6 or more. For loads perpendicular to the grain, the maximum efficiency occurs at an L/D ratio of 8 and decreases for smaller or larger L/D ratios.

Staggering bolts should be avoided when the load acts parallel to the grain. If an odd bolt is required, the usual practice is to alter the diameter of all the bolts so that an even number can be used. Moreover, if an odd bolt is used and placed in the center of the timber, it must be considered as being between the last pair of bolts in computing the critical section, or it must be placed at a distance from them greater than the center-to-center spacing. For loads acting perpendicular to the grain, staggering is preferable to avoid splitting.



Courtesy Timber Engineers' Handbook, Edited by H. J. Hansen

Fig. 41. Values of L for L/D ratios in joint of four members of equal thickness.

Seasoning after assembly may start splitting at the bolts, and for this reason cross bolts are recommended. The tests conducted at the Forest Products Laboratory have not been sufficient to furnish rules for the size and number of cross bolts to use under various conditions. However, cross bolts can be smaller in diameter than the main bolts in a joint and often only one or two are required. In a tension joint the cross bolts should be placed in the end distance, and in a joint with the load perpendicular to the grain the cross bolts should be placed in the main member just outside the main bolt group.

It is impossible to present general rules covering the spacing and end and edge distances for bolts when the load is applied at an angle to the grain. It is important, however, to arrange the bolts so that the center of resistance of the bolt group passes through the intersection of the gravity axis of the members.

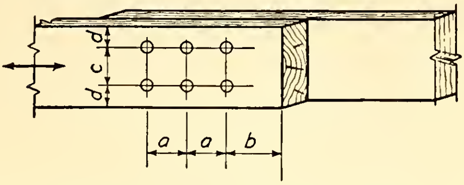


FIG. 42. Method of measuring spacing parallel to grain.

a = Center-to-center spacing. Minimum four times bolt diameter.

b = End distance. In tension, 7 times bolt diameter for softwoods and 5 times bolt diameter for hardwoods. In compression, 4 times bolt diameter.

c = Row spacing. Determined by net area at critical section. This area must be at least 80 per cent of total area in bearing under all the bolts.

d = Edge distance. At least 1½ times bolt diameter for L/D ratios of 5 or 6. For ratios greater than 6 increase slightly, and for ratios less than 5 decrease slightly. A ratio of 2:1 between row spacing and edge distance is preferable.

Of particular interest to the designing engineer using bolts should be the subject of camber. As a general rule bolts are inserted into holes slightly larger than their diameter, which naturally increases the yield of a timber structure. For this reason the engineer should provide ample

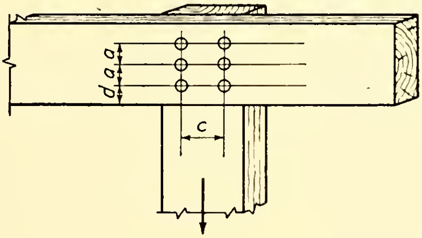


FIG. 43. Method of measuring spacing perpendicular to grain.

a = Center-to-center spacing. Four times bolt diameter, unless design load is much less than bolt bearing capacity of side members. In this case spacing may be reduced. When metal splice plates are used, spacing required is sufficient space to tighten nuts.

b = No end distance in this type of joint.

c = Row spacing. Minimum, two and one-half times bolt diameter for L/D ratios of 2, and 5 times bolt diameter for L/D ratios of 6 or greater.

d = Edge distance. Minimum, four times bolt diameter. In compression the edge distance would be at the opposite edge from that shown in Fig. 43.

cambers for bolted structures and should keep to a minimum the number of joints and splices.

102. Metal Splice Plates. When metal splice plates are used the stresses induced in the plates and the bearing of the bolt on the plates should not exceed the allowable stresses for the metal used.

The thickness of metal plates can be determined from the following formula:

$$t = \frac{P}{ns[d - h_n(D + \frac{1}{16})]}$$

where t = thickness of plate in inches.
P = load in pounds.
n = number of plates.
s = allowable working stress of metal in pounds per square inch.
d = depth of main member in inches.
h_n = number of holes at critical section.
D = diameter of bolt in inches.

ILLUSTRATIVE PROBLEMS

Problem. Find the number and size of lag screws and bolts for the tension joint in Fig. 44. Use seasoned shortleaf southern pine.

Lag Screws. The penetration into the member receiving the point should be 9D. Select a screw with a diameter of 3/8 in. This makes the total length = 9 × 3/8 + 3.625 = 7 in. For this length the threaded portion of the screw, from Table 18, is 4 in. This leaves a shank 3 in. long, which makes it necessary to reduce the general equation because the shank does not extend to the plane of contact between the two members.

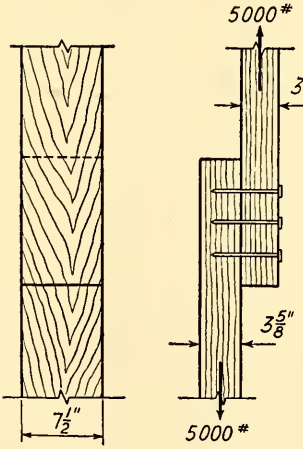


FIG. 44. Tension lap joint.

The percentage to use as a multiplier in the equation may be found as follows:

$$\text{Percentage} = 100 - 20\frac{(t_1 + t_2 - L_1)}{t_2}$$

where t_1 = thickness of washer.
t_2 = thickness of cleat.
L_1 = length of shank.

$$\text{Percentage} = 100 - 20\left(\frac{0.0781 + 3.625 - 3}{3.625}\right) = 96.2 \text{ per cent}$$

The ratio of cleat thickness to the diameter of screw is 3.625/0.375 = 9.7, and according to Chart 12 the safe load can be increased by 25 per cent.

$$\text{Required penetration} = \sqrt{1.25 \times 0.962 \times 9D} = 11D$$

Actual penetration = $9D$

Percentage to use as a multiplier of safe load = $\frac{9}{11} \times 1.25 \times 0.962 = 98.5$ per cent

Safe load: $P = 2280 \times 0.985 \times 0.375^2 = 316$ lb.

$$\text{Number of lag screws required} = \frac{5000}{316} = 16$$

Bolts. The length of bolt for calculating the safe load is $2 \times 3.625 = 7.25$. Using bolts with a diameter of 1¼ in. the L/D ratio is 5.8; for this L/D ratio Chart 15 shows a working stress of 1100 lb. per sq. in.

$$\text{Safe load} = 1100 \times 0.8 \times 0.5 \times 1.25 \times 7.25 = 4000 \text{ lb.}$$

$$\text{Number of bolts required} = \frac{5000}{4000} = 2$$

Problem. Find the number and size of lag screws and bolts for the lap joint in Fig. 45. Use seasoned shortleaf southern pine.

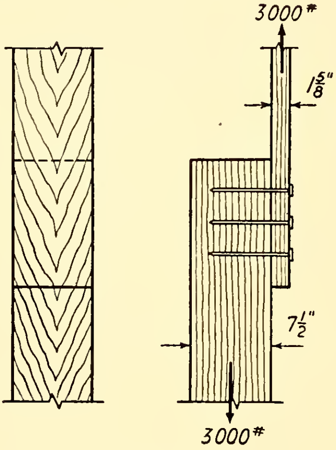


FIG. 45. Tension lap joint.

Lag Screws. Use ½-in. lag screws with a length of 6½ in. For this length the shank is 3 in. and extends 1⅜ in. into the 8-in. timber. The ratio of the length of shank in the 8-in. member to the diameter is 2.75, which makes it possible to increase the load by 24 per cent (Chart 13).

The ratio of cleat thickness to the diameter is 3.25 so it is necessary to reduce the load by 4 per cent (Chart 12).

Required penetration = $\sqrt{1.24} \times \sqrt{0.96} \times 9D = 9.9D$

Actual penetration = $7D$

Percentage to use as a multiplier of safe load = $\frac{7}{9.9} \times 1.24 \times 0.96 = 84$ per cent

Safe load for one screw = $2280 \times 0.84 \times 0.5^2 = 478$ lb.

$$\text{Number of screws required} = \frac{3000}{478} = 7$$

Bolts. The length of the bolt for calculating the safe load is $2 \times 1.625 = 3.25$. Use ½-in. bolts. $L/D = 6.5$.

$$\text{Safe load for one bolt} = 1000 \times 0.8 \times 0.5 \times 3.25 \times 0.5 = 648 \text{ lb.}$$

$$\text{Number of bolts required} = \frac{3000}{648} = 5$$

Remarks. A comparison of this problem with the previous one brings out the fact that the thickness of cleat used has a bearing on the load-carrying capacity of lag screws. It is preferable to use larger diameters when thick cleats are used in the joint, making the ratio of cleat thickness to the diameter of the lag screw approximately 3.5.

Problem. Design a bolted tension joint to carry 55,000 lb. Use Douglas fir (coast region), and assume that the lumber is seasoned and the joint is to be used in a dry inside location.

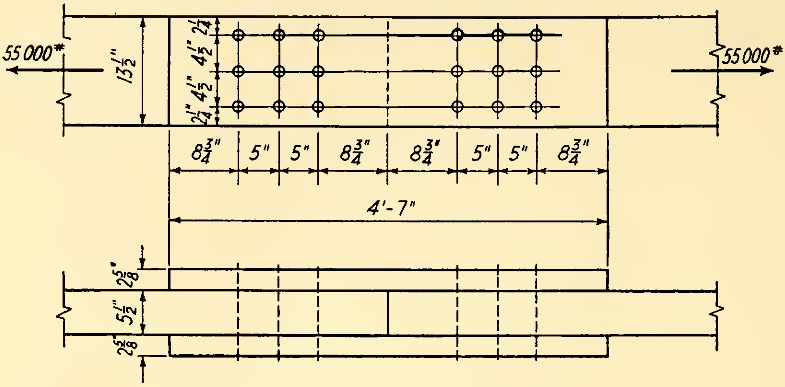


Fig. 46. Detail of bolted tension joint.

Bolts. Assume a main member 6 in. thick with 3-in. splice plates. Try 1¼-in. bolts. $L/D = 5.5/1.25 = 4.4$.

For an L/D ratio of 4.4, the working stress (Chart 15) is 1360 lb. per sq. in.

$$\text{Safe stress with wood splice plates} = 1360 \times 0.8 = 1088 \text{ lb. per sq. in.}$$

$$\text{Safe load for one bolt} = 1088 \times 1.25 \times 5.5 = 7500 \text{ lb.}$$

$$\text{Number of bolts required} = \frac{55{,}000}{7500} = 8 \quad \text{(use 9)}$$

Try a 6 by 14-in. member with area = 74.25 sq. in. Bearing under all bolts = 9(1.25 × 5.5) = 61.9 sq. in. Required area equals 80 per cent of bearing area under all bolts plus bearing area of number of bolts in a row.

$$\begin{aligned} \text{80 per cent of bearing area} &= 61.9 \times 0.8 = 49.5 \\ \text{With 3 bolts in a row} &= 3(1.25 \times 5.5) = 20.6 \\ \hline \text{Required area} &= 70.1 \text{ sq. in.} \end{aligned}$$

A 6 by 14 in. member will satisfy.

Problem. Find the size and number of bolts required for a two-member joint with an 8000-lb. load acting in tension at an angle of 60°. Use dry shortleaf southern pine with metal splice plates. Detail the joint.

Assumptions. Members will be 4 in. thick, and bolts will have a diameter of ½ in.

The L/D ratio is

$$\frac{L}{D} = \frac{3.625}{\frac{1}{2}} = 7.25$$

Safe load for one bolt

Parallel to grain: 860 × 0.5 × 3.625 = 1560 lb.
Perpendicular to grain: 315 × 1.68 × 0.5 × 3.625 = 960 lb.

$$\text{At angle of } 60°\text{: } \frac{1560 \times 960}{1560 \times 0.75 + 960 \times 0.25} = 1060 \text{ lb.}$$

Member A:

$$\text{Number of bolts required} = \frac{8000}{1060} = 8$$

Member B:

$$\text{Number of bolts required} = \frac{8000}{1570} = 6$$

Size of member B. Consider 2 rows of bolts.

80 per cent of bearing under all bolts = 0.8 × 6(0.5 × 3.625) = 8.7
Bearing area of 2 bolts at critical section = 2(0.5 × 3.625) = 3.625

Required area = 12.325 sq. in.

A 4 by 4 in. member with an area of 13.14 sq. in. will satisfy.

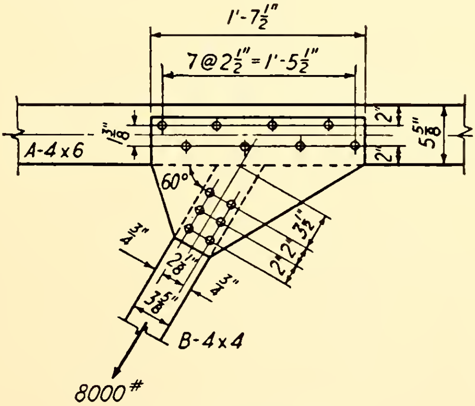


Fig. 47. Detail of joint.

Problem. Find the size and number of lag screws for the joint shown in Fig. 48. Use seasoned Douglas fir.

Lag screws. Use ½-in. lag screws.

Length = 9 × ½ = 4.5 + 2.625 = 7.125 in. (Use 8 in.)

Ratio of depth of shank in main member to diameter of screw = 0.875/0.5 = 1.75. This allows an increase in safe load of 15 per cent.

Ratio of cleat thickness to diameter of screw = 2.625/0.5 = 5.25. This allows an increase in safe load of 19 per cent.

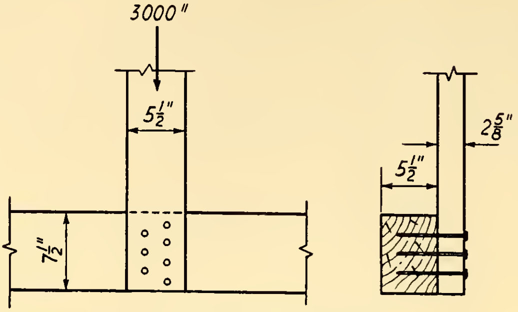


Fig. 48. Lag-screw joint.

Required penetration $= \sqrt{1.15 \times 1.19} \times 9D = 10.5D$

Actual penetration $= 9D$

Percentage to use as multiplier of safe load $= \frac{9}{10.5} \times 1.15 \times 1.19 = 117$ per cent

Safe load parallel to grain $= 2280 \times 0.5^2 \times 1.17 = 667$ lb.

Safe load perpendicular to grain $= 667 \times 0.68$ (from Table 20) $= 453$ lb.

Number of lag screws required $= \frac{3000}{453} = 7$

CONNECTORS

103. General. Probably no single factor is more responsible for revolutionizing timber design than the development of modern timber connectors. The results of tests on many types of connectors were first reported in this country in 1933 by Nelson S. Perkins, Peter Landsem, and G. W. Trayer. These tests formed the basis for assigning safe working loads to connectors when used with native woods. In 1934 the Timber Engineering Company of Washington, D. C., a subsidiary of the National Lumber Manufacturer's Association, acquired the patent rights on a number of these connectors for the purpose of distributing them on a commercial basis. Since then, many additional tests and refinements in manufacturing have been made.

Timber connectors are essentially metal rings or plates that are embedded partly in each face of adjacent members to transmit the load from one to the other. They are used in combination with bolts of small diameter. Connectors enlarge the bearing area of the joint stress.

thereby often making it possible to develop the full allowable load of the members connected.

104. Types of Connectors. More than 60 different types of connectors have been patented in the United States and Europe. Among the most commonly used in this country are the split ring, toothed ring, shear and claw plates, spike grids, and clamping plates.

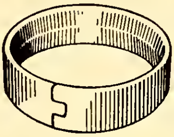


FIG. 49. Split ring.

Manufactured from low-carbon steel. Used between two timber faces for heavy construction and fits into pre-cut grooves in the timber faces. The tongue and groove "split" permits simultaneous ring bearing against the core wall and outer wall of the groove into which the ring is placed. The inside bevel and mill edge facilitate installation into and removal from its groove.

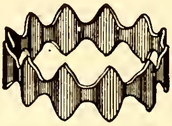


FIG. 50. Toothed ring.

Manufactured from low-carbon steel. Used between two timber faces for comparatively light construction and embedded into the contact faces of the joint members by means of pressure.

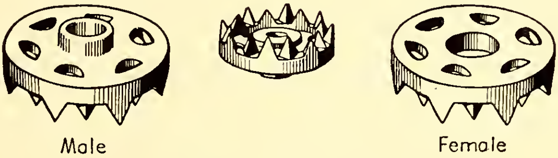


FIG. 51. Claw plates.

Malleable iron connectors. Used as "units" either in pairs for timber-to-timber connections or singly in making timber-to-metal connections. The female plates are adapted to use when the connector must lie flush with the surface of the timber. Claw plates are installed by forcing the teeth into the wood beyond the depth of the circular dap cut to receive the rim and plate portions.

105. Sizes. The connectors illustrated in the previous article are manufactured in several different sizes. The split and toothed rings, claw plates, and shear plates are round, and their size is specified by diameter. The spike grids are square, and the length of one side of the square gives the size. Clamping plates are manufactured with and without flanges. Those without flanges are usually square and those with flanges are rectangular. Table 25 lists the dimensions of the types of connectors most commonly used.

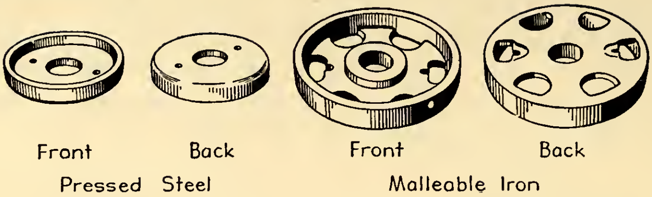


Fig. 52. Shear plates.

These plates when installed lie flush with the timber surface. They are used as "units" in pairs for timber-to-timber joints with two plates placed back to back or singly in timber-to-metal joints with the plate placed with its back toward the metal. The plates fit into pre-cut grooves in the timber faces.

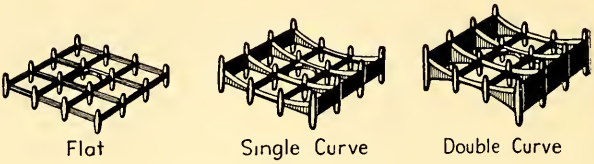


Fig. 53. Spike grids.

Manufactured from malleable cast iron. Used primarily in pier and trestle construction between either flat or curved surfaces. They are embedded into the wood surfaces by means of pressure.

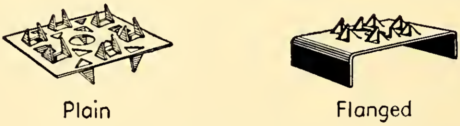


Fig. 54. Clamping plates.

Stamped from metal sheets. Used as railroad "tie spacers" between ties and guard timbers to keep the ties properly spaced or where timbers overlap at right angles. The plain clamping plate with teeth on opposite faces is seated by means of a special block which protects the connector during the driving process. The flanged clamping plate with teeth on one face only is driven into place with a maul or ram, the connector being protected from damage by a steel cover plate.

Table 25. Connector Sizes

[Dimensions in inches]

Timber Connector		Shipping Weight (Pounds per 100)	Size	Dimensions of Metal		Groove Size		
				Depth	Thickness	Inside Diameter	Width	Depth
Split rings		30	$2\frac{1}{2}$	0.75	0.163	2.56	0.18	0.37
		76	4	1.00	0.193	4.08	0.21	0.50
Toothed rings		9	2	0.94	0.061			
		12	$2\frac{5}{8}$	0.94	0.061			
		15	$3\frac{3}{8}$	0.94	0.061			
		18	4	0.94	0.061			
Claw plates		58	$2\frac{5}{8}$	0.75		2.14	0.25	0.38
		70	$3\frac{1}{8}$	0.75		2.66	0.25	0.38
		93	4	0.75		3.57	0.25	0.38
Shear plates	pressed steel	40	$2\frac{5}{8}$	0.375	0.169	2.25	0.19	0.45
	malleable iron	100	4	0.62	0.20	3.49	0.27	0.64
Spike grids	flat	50	$4\frac{1}{8} \times 4\frac{1}{8}$	1.00				
	single curve	75	$4\frac{1}{8} \times 4\frac{1}{8}$	1.38				
	double curve	107	$4\frac{1}{8} \times 4\frac{1}{8}$	1.75				
Clamping plates	plain	65	$5\frac{1}{4} \times 5\frac{1}{4}$		0.077			
	flanged	200	5×8	2.00	0.122			

106. Theoretical Action of a Split Ring. Although considerable progress has been made in the theoretical stress analysis of connector joints, the stress distribution is so complicated that actual tests must be relied upon to provide the necessary design data. However, an analysis of the stresses in a connector joint does provide a check on test results and aids in the interpretation of test data.

The primary stresses in the wood of a tension joint are illustrated in Fig. 55 and may be classified as shear, compression, and tension. The shaded areas represent the part of the wood in shear A, compression B and C, and tension D.

In a tension joint with two split-ring connectors in opposite faces of the center member and the load applied parallel to the grain of the wood, the shear, compression, and tension areas may be expressed as follows: *

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* "Timber Connector Joints: Their Strength and Design," by J. A. Scholten, *Tech. Bull.* 865, U. S. Dept. Agr., 1944.

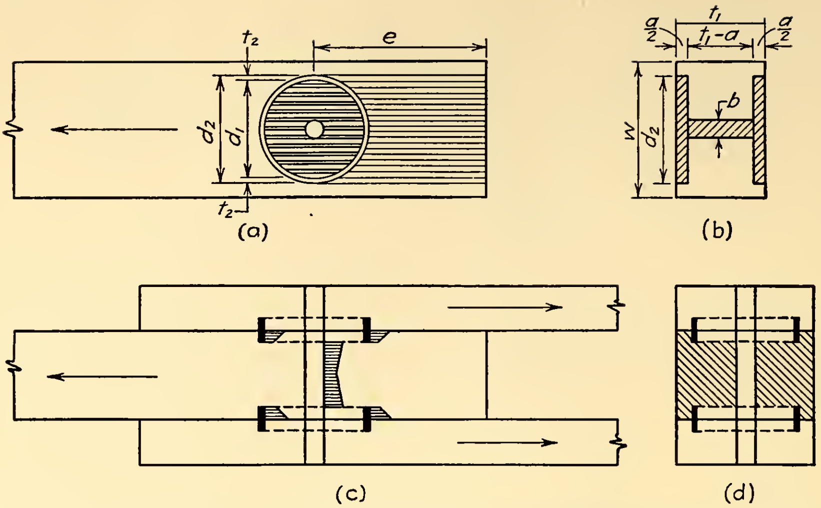

Fig. 55. Detail of a connector joint, showing portions of center member subjected to shear (*a*), compression (*b*, *c*), and tension (*d*).

Shear area:

$$\text{Within core} = 2\left(\frac{\pi d_1{}^2}{4}\right)$$

$$\text{Below core} = 2\left[d_2e - \frac{1}{2}\left(\frac{\pi d_2{}^2}{4}\right) + 2\left(\frac{ae}{2}\right)\right]$$

Compression area:

$$2\left(\frac{ad_2}{2}\right) + b(t_1 - a)$$

Tension area:

$$t_1w - \left[2\left(\frac{ad_2}{2}\right) + b(t_1 - a)\right]$$

where d_1 = inside diameter of core.
d_1 = outside diameter of core.
e = end distance.
a = depth of connector.
b = diameter of bolt.
t_1 = thickness of member.
w = width of member.
t_2 = thickness of metal.

Because of the interaction between the wood and metal, the presence of irregularities in the wood itself, and the nonhomogeneous structure of wood, it is impossible to apply the theory of elasticity to the distribution of the stresses in a connector joint. Many factors, related to the properties of the material, affect the compression, shear, and tensile stresses in a connector joint.

In joints subjected to tension, the concentration of stress in the member due to abrupt changes in the continuity of the grain causes failure in tension at a stress approximately equal to the compressive stress of clear wood. Therefore, the total tension area, as shown in Fig. 55*d*, of the member at the critical section should not be stressed in excess of the safe compressive stress parallel to the grain for clear material.

In compression, the projected area of the connector and the bolt multiplied by the maximum compressive strength of the wood might be expected to give the maximum load. However, the maximum compressive strength of the wood is not usually developed over the full projected area. If the projected area under a split-ring connector in compression parallel to the grain is multiplied by the maximum crushing strength of the wood, the load obtained varies from 65 to 100 per cent of the maximum test load depending upon the size of the connector. If friction is neglected, and the remaining portion of the load is assumed to be carried by the bolt, the average stress developed under the bolt varies from 25 to 100 per cent of the maximum crushing strength of the wood. The larger percentages apply for thin members and the smaller percentages for thick members. Often the bolt never reaches its maximum capacity because the slip in the connector joint at the maximum load is less than that needed to develop the maximum compressive strength of the wood under the bolt.

In shear, the different sizes of split-ring connectors give varying results which can only be attributed to the relative amounts of wood in shear and compression. The ratio of the core area in shear to the area in compression under the connector will vary considerably for the different sizes of connectors. The load sustained in shear does not depend solely on the shear area. The actual test loads do not increase in proportion to the size of the shear area.

107. Testing Procedure. Recent tests * on connectors indicate that many factors affect the strength of the joint. In addition to the size of the connector these factors are (1) the species of the wood, (2) the direction of the load with respect to the grain of the wood, (3) the thickness and width of the wood, (4) the moisture content, (5) the presence of checks, (6) the edge and end distances, (7) the spacing of connectors, (8) the size of the ring groove, (9) the position of the tongue and slot of the split-ring with respect to the direction of the grain, and (10) the size of the bolt hole.

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* "Timber Connector Joints; Their Strength and Design," by J. A. Scholten, *Tech. Bull.* 865, U. S. Dept. Agr., 1944.

Two general types of tests were conducted. In one the load was applied parallel to the grain, and in the other the load was applied perpendicular to the grain. The test specimen consisted of three members with two connectors and a bolt, except when four connectors were used to determine the effect of spacing. Control specimens cut from each member were tested to determine the strength characteristics of the material.

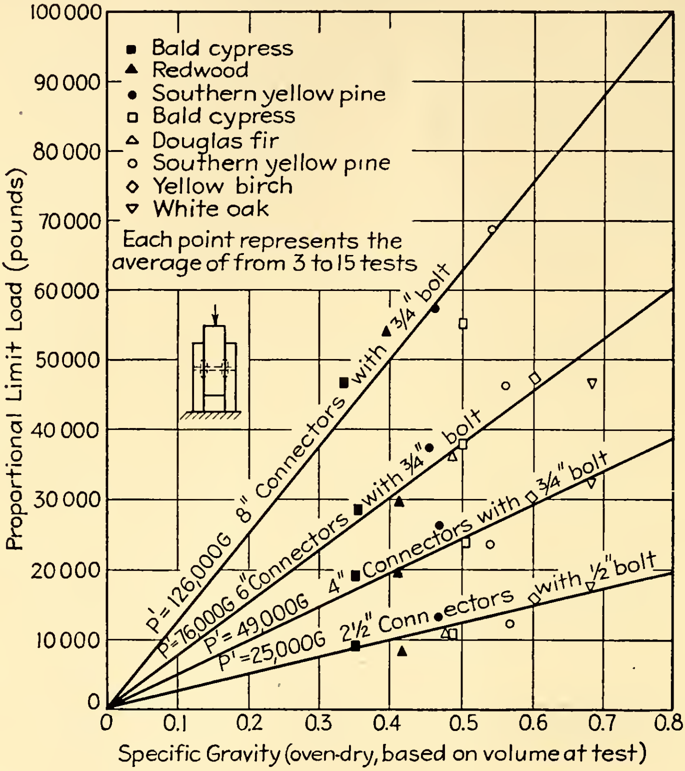

Chart 17. Relation between proportional limit load bearing parallel to the grain and specific gravity of air-dry wood for split-ring connector joints consisting of two connectors and a single bolt. The solid and open symbols of the same species indicate marked differences in specific gravity.

Load was applied continuously, and the rate of descent of the testing machine was within the range of 0.283 to 0.382 in. per minute. The slip in the joint was determined with dial gages graduated to 0.001 in.

Four types of split-ring connectors, in addition to other types, were tested. However, the 2½- and 4-in. split rings are most commonly used in practice.

Although a number of different properties of wood determine the resistance to lateral displacement of connectors, the tests show that the

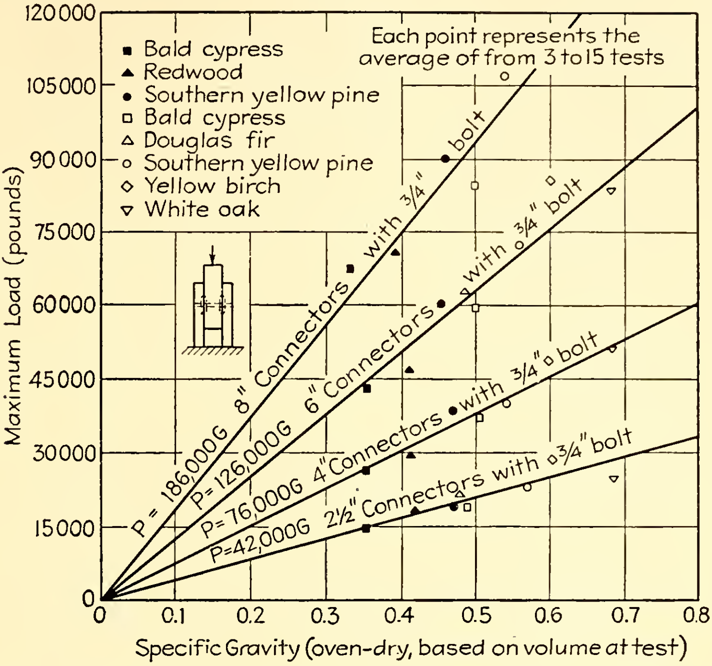

CHART 18. Relation between maximum load bearing parallel to the grain and specific gravity of air-dry wood for split-ring joints consisting of two connectors and one bolt. The solid and open symbols for the same species indicate marked differences in specific gravity.

specific gravity of the wood is the best criterion for strength. Both the proportional limit loads and maximum loads increase directly with specific gravity as illustrated in Charts 17 and 18.

The proportional limit load is approximately one-half to two-thirds of the maximum load depending upon the species of wood and the size of the connector. (See Chart 19.)

For the 2½-in. connector the loads at first drop were about 25 per cent higher than the proportional limit loads and about 75 per cent of the maximum loads. For the larger connectors the loads at first drop approached the maximum loads. After the core within the connector sheared off, the split-ring connectors continued to function as metal bands around solid cores until final failure, for the smaller connectors, which consisted of bending of the bolt and crushing of the wood on the faces of the members under the connectors and the bolt.

In the tests with loads perpendicular to the grain, the loads were found to be 58 per cent of the parallel-to-grain values for 2½- and 4-in. split rings and 50 per cent for 6-in., when the width of the member was 3⅝, 5½, and 7½ in. respectively.

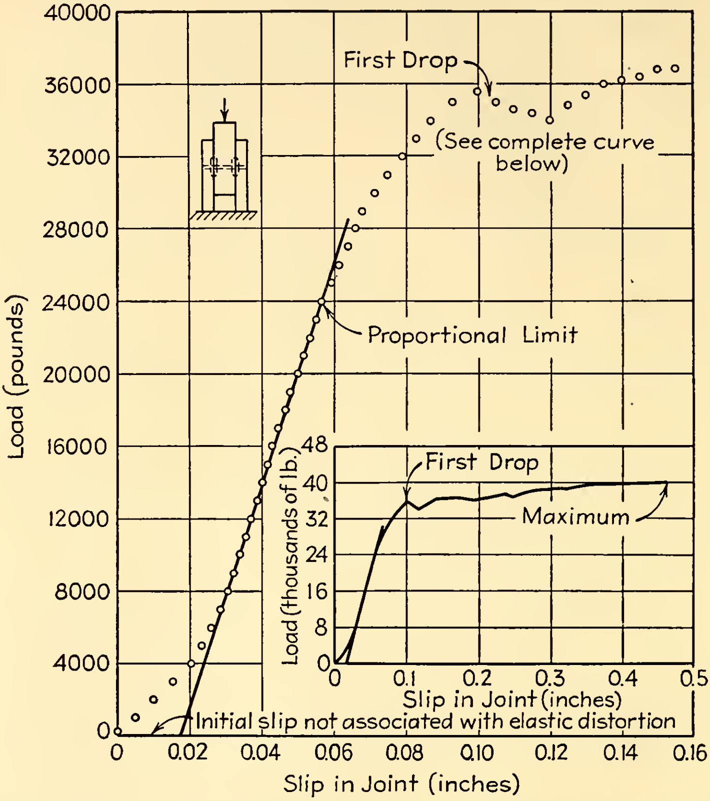

CHART 19. Relation between load and slip in split-ring connector joint for an individual test specimen; bearing parallel to the grain in air-dry wood.

The relation between the load and thickness of the main member in a three-member joint with the load parallel to the grain is shown in Chart 20, and the relation between the load and the width of the timber when the load is perpendicular to the grain is shown in Chart 21.

On the basis of the results of tests illustrated in Charts 20 and 21 and additional tests on bolts it has been possible to set up optimum thicknesses and required widths for the timber,

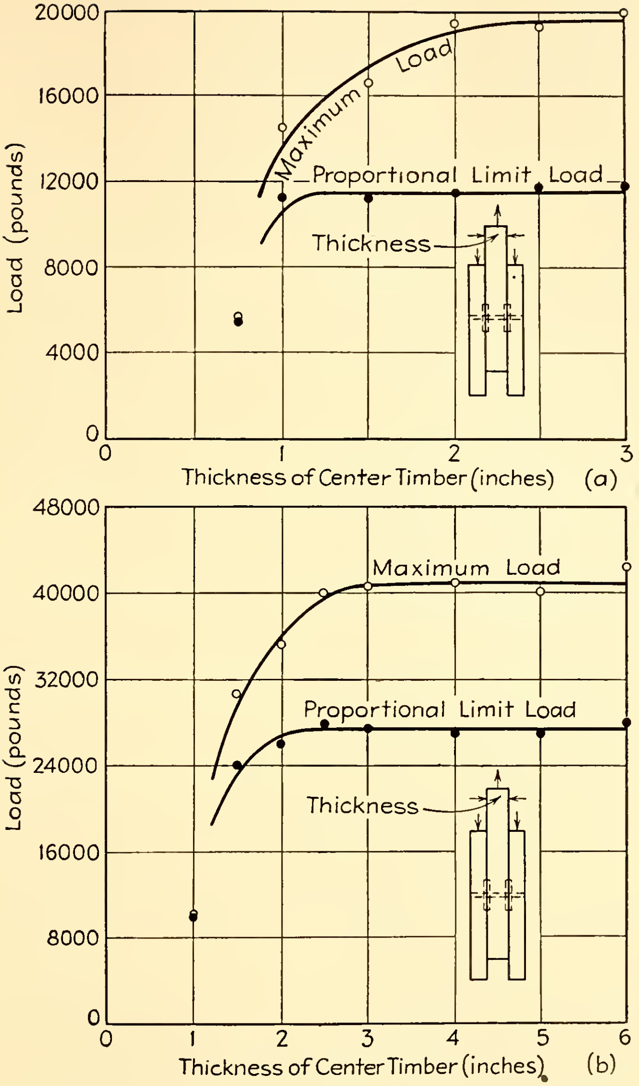

CHART 20. Relation between load and thickness of timber for three-member split-ring connector joints bearing parallel to the grain of air-dry southern pine (*a*) with 2½-in. connectors, and (*b*) with 4-in. connectors.

From the analysis of these tests of three-member joints, it is shown that a 2-in. thickness of member is required to develop the optimum load with a 2½-in. connector and a ½-in. bolt, and a 3-in. thickness is required with a 4-in. connector and a ¾-in. bolt. When connectors are placed in only one face of a member, the thickness required to develop the optimum load for one connector is one-half of the above values plus ⅛ in.

When the load is less than the optimum the thickness should not be reduced below a certain minimum. When 2½- or 4-in. connectors are used in pairs, the minimum thickness should be ½ in. greater than the sum of the depths of the two connectors. If the connectors are placed in one face only, the grooves for the 2½- and 4-in. connectors should lack ½ in. of penetrating the piece in determining the minimum thickness. The safe loads at these minimum thicknesses should be about two-thirds of the optimum load.

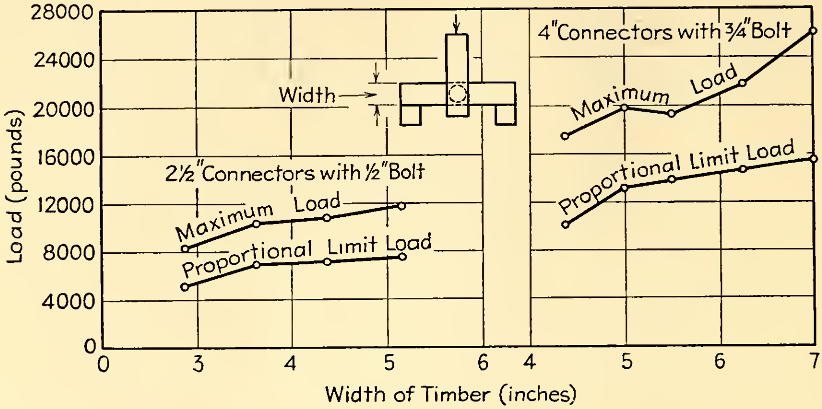

CHART 21. Relation between load and width of timber for three-member split-ring connector joints bearing perpendicular to the grain of air-dry southern pine.

The minimum widths of lumber for loads parallel to the grain with 2½- and 4-in. connectors should be 3⅝ and 5½ in. respectively. The results of tests show that an increase in width over these minimum values is accompanied by a very small increase in maximum load, and the increase in proportional limit loads is negligible.

Tests to determine the effect of width of member on the strength of joints bearing perpendicular to the grain with 2½- and 4-in. connectors indicated that, with transverse chord members equal in width to the outside diameter of the connectors, the loads averaged about three-fourths of the loads in widths 3⅝ and 5½ in. respectively. It is recommended, however, that the minimum widths of 3⅝ and 5½ in. for 2½- and 4-in. connectors be used.

The edge distance, measured from the center of the connector to the edge of the piece, has a significant influence on the strength of the joint when the load is bearing perpendicular to the grain. The results of the tests show that minimum values should be one-half the thickness of 3⅝- and 5½-in. members for 2½- and 4-in. connectors for either edge; that is, the edge of the member toward which the load is acting or the opposite edge.

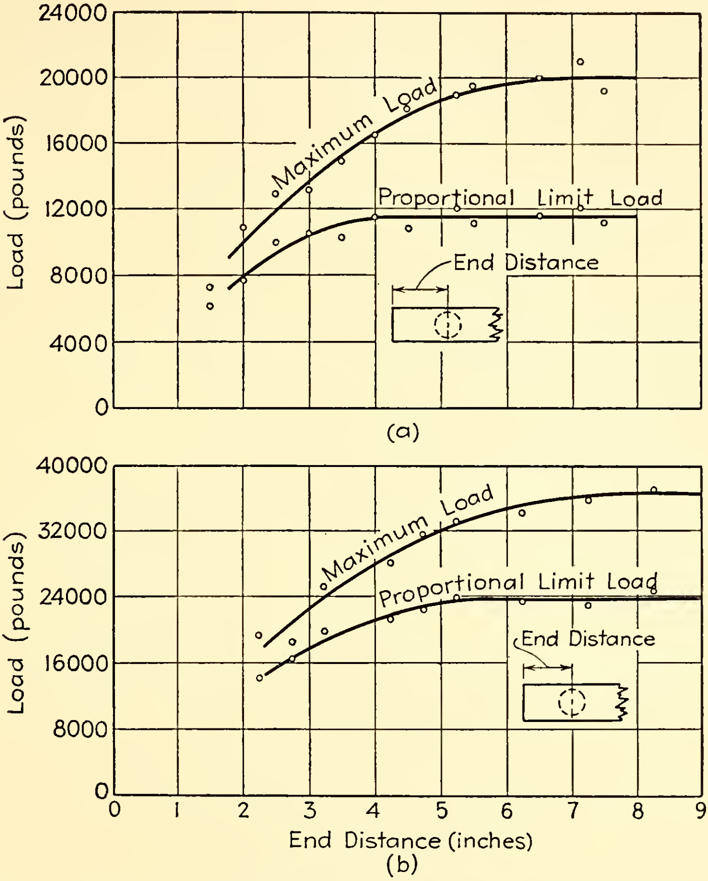

CHART 22. Relation between load and end distance for a split-ring joint bearing in tension parallel to the grain of air-dry southern pine. (*a*) Two 2½-in. connectors and a ½-in. bolt, and (*b*) two 4-in. connectors and a ¾-in. bolt.

The end distance, measured from the center of the connector to the end of the timber, has considerable influence on the strength of the joint when the load is parallel to the grain. Chart 22 shows the proportional limit and maximum loads for 2½- and 4-in. connectors with different end distances. The results of the tests and an analysis of compression and shear stresses in the joints made of seasoned unchecked material show that the end distance required to produce a maximum load is equal to the nominal diameter of the connector plus 3 in. When the end distance is less, the safe working loads are naturally lower. The tests show that the load is reduced uniformly from unity to 62.5 per cent at half the end distance producing the maximum load. End distances less than half the maximum values are not recommended.

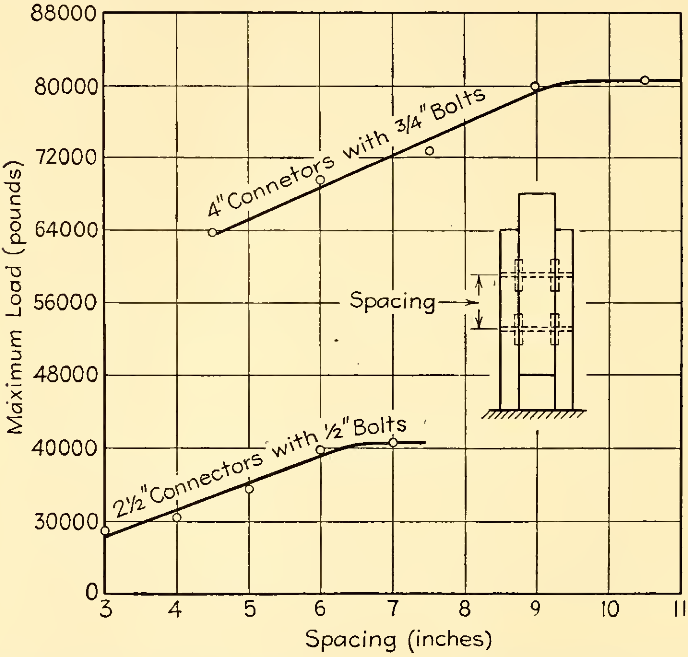

Chart 23. Relation between maximum load and longitudinal spacing for a split-ring joint consisting of four connectors and two bolts bearing parallel to the grain of air-dry southern pine.

Chart 23 is a typical load-spacing diagram. The results of tests and an analysis of the stresses in the member at the joint show that the center-to-center spacing of connectors required to give the maximum loads is 1½ times the nominal diameter of the connector plus 3 in. For spacings less than this the safe working loads for the second pair of connectors should be reduced uniformly from unity at the maximum spacing to 50 per cent at a spacing equal to the nominal diameter of the connector plus ⅞ in. for 2½- and 4-in. connectors.

108. Bolts and Washers. The design loads for connectors are based on a minimum size of bolts and washers. Table 26 lists the minimum sizes to use with various connectors in order to develop their standard design loads.

Table 26. Sizes of Bolts and Washers Used with Connectors

[Dimensions in inches]

Timber Connector		Size	Bolt	Round Washers			Plate Washers	
				Cast or Malleable Iron	Wrought Iron			
				Diameter	Diameter	Thickness	Size	Thickness
Split rings		$2\frac{1}{2}$	$\frac{1}{2}$	$2\frac{1}{8}$	$1\frac{3}{8}$	$\frac{3}{32}$	2	$\frac{1}{8}$
		4	$\frac{3}{4}$	3	2	$\frac{5}{32}$	3	$\frac{3}{16}$
Toothed rings		2	$\frac{1}{2}$	2			2	$\frac{3}{16}$
		$2\frac{5}{8}$	$\frac{5}{8}$	$2\frac{5}{8}$			$2\frac{1}{2}$	$\frac{1}{4}$
		$3\frac{3}{8}$	$\frac{3}{4}$	3			3	$\frac{1}{4}$
		4	$\frac{3}{4}$	$3\frac{1}{2}$			$3\frac{1}{2}$	$\frac{3}{8}$
Claw plates	male	$2\frac{5}{8}$	$\frac{1}{2}$	$2\frac{1}{8}$	2	$\frac{5}{32}$	2	$\frac{3}{16}$
	female	$2\frac{5}{8}$	$\frac{7}{8}$	$3\frac{1}{2}$	2	$\frac{5}{32}$	3	$\frac{1}{4}$
	male	$3\frac{1}{8}$	$\frac{1}{2}$	$2\frac{1}{8}$	2	$\frac{5}{32}$	2	$\frac{3}{16}$
	female	$3\frac{1}{8}$	$\frac{7}{8}$	$3\frac{1}{2}$	2	$\frac{5}{32}$	3	$\frac{1}{4}$
	male	4	$\frac{3}{4}$	3	4	$\frac{11}{64}$	3	$\frac{3}{8}$
	female	4	$1\frac{1}{4}$	$4\frac{3}{8}$	4	$\frac{11}{64}$	4	$\frac{3}{8}$
Shear plates	pressed steel	$2\frac{5}{8}$	$\frac{3}{4}$	3	2	$\frac{5}{32}$	3	$\frac{1}{4}$
	malleable iron	4	$\frac{3}{4}$	3	2	$\frac{5}{32}$	3	$\frac{1}{4}$
	malleable iron	4	$\frac{7}{8}$	$3\frac{1}{2}$	$2\frac{1}{4}$	$\frac{11}{64}$	3	$\frac{1}{4}$
Spike grids	flat	$4\frac{1}{8} \times 4\frac{1}{8}$	$\frac{3}{4}$ or 1	3 or 4			3	$\frac{3}{8}$
	single curve	$4\frac{1}{8} \times 4\frac{1}{8}$	$\frac{3}{4}$ or 1	3 or 4			3	$\frac{3}{8}$
	double curve	$4\frac{1}{8} \times 4\frac{1}{8}$	$\frac{3}{4}$ or 1	3 or 4			3	$\frac{3}{8}$
Clamping plates	plain	$5\frac{1}{4} \times 5\frac{1}{4}$	$\frac{3}{4}$	3			3	$\frac{1}{4}$
	flanged	5×8	$\frac{3}{4}$	3			3	$\frac{1}{4}$

The allowable connector loads also assume that the faces of adjacent members remain in contact. When the lumber is not seasoned to the moisture content that it will ultimately attain in service, the joints should be tightened periodically until moisture equilibrium is reached.

109. Safe Loads. The safe loads for connectors are based on the species and thickness of lumber, the end and edge distances, the spacing of the connectors, and the direction of the load with respect to the grain. The safe loads for split-ring connectors are given in Charts 24 and 25. Values for other connectors may be obtained from the Manual of Timber Connector Construction published by the Timber Engineering Company.

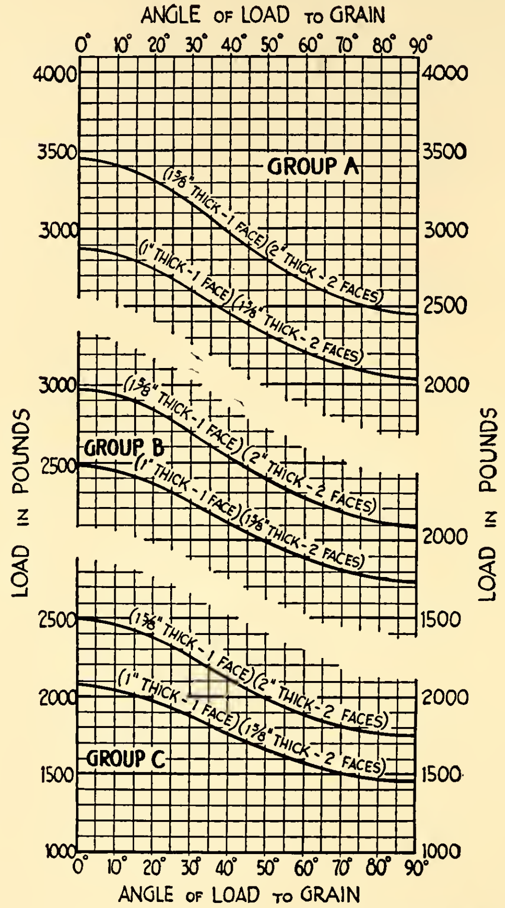

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Chart 24. Safe loads for one 2½-in. split ring and bolt in single shear.

For joints fully loaded for many years, use 90 per cent of the chart values. Since wood is able to support much greater loads when the loads are applied for a short period, the values from the charts may be increased under short-time loading conditions. The permissible increases are given in Table 27.

Table 27. Permissible Increases in Connector Loads for Load Duration

[Values given in percentage]

	Permanent Loading	*Three-Month Loading*	*Snow Loading*	*Wind or Earthquake Loading*	*Impact Loading*
Split rings	0	15	15	33⅓	100

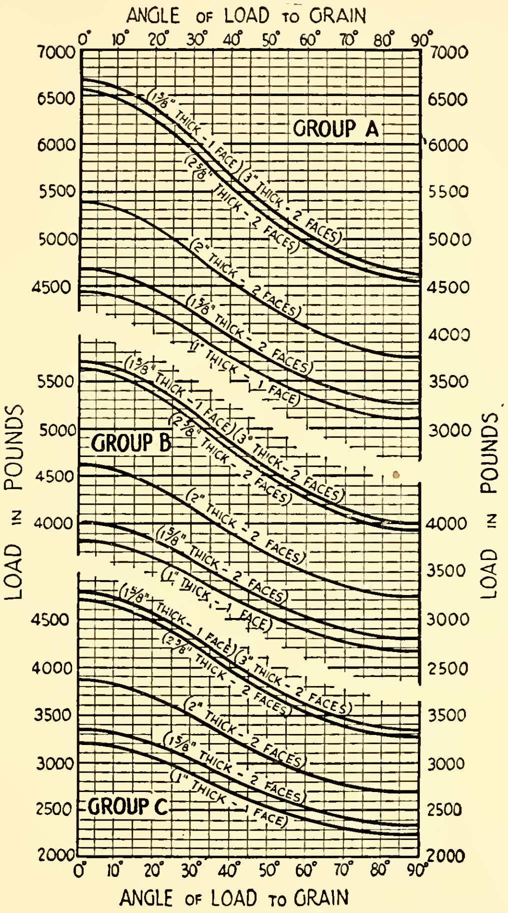

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CHART 25. Safe loads for one 4-in. split ring and bolt in single shear.

Loading of three months' duration is loading not continuously applied for periods exceeding three months, and such loading in combination with permanent loading when the permanent loading does not exceed 25 per cent of the total combined load.

Snow loading refers to the loading when maximum snow is intermittently but not permanently applied.

The increases for maximum wind or earthquake apply to this type of loading or any other loading not exceeding 5 minutes' duration. For wind in combination with permanent loading, the resulting sizes and number of connectors should not be smaller than those required for permanent loading above.

Impact loading applies only to that portion of the load producing impact.

Usually the long-time or permanent load values will govern a design because the increase in connector load capacity is most generally in greater proportion than the increase in load.

The allowable connector loads for angles of load to grain between 0° and 90° have been determined by the Hankinson formula given in Chapter III.

The allowable connector loads are divided into groups depending upon the species used and apply to all stress grades of a given species except where modifications for density are made as indicated in Table 28.

Table 28. Connector Load Grouping of Species[1] When Structurally Graded

Connector Load Grouping		
Group A Species	*Group B* Species	*Group C* Species
Douglas fir (dense)	Douglas fir (coast region)	Cypress, southern and tidewater red
Oak, red and white	Larch, western	Hemlock, West Coast
Pine, southern (dense)	Pine, southern	Pine, Norway
		Redwood

[1] For other species, information may be obtained from the Timber Engineering Company.

110. Condition of Lumber. The allowable connector loads are based on lumber which is seasoned to approximately 15 per cent moisture content to a depth of ¾ in. from the surface before fabrication and which will remain dry in service.

For connectors in lumber which is fabricated before it is seasoned to a depth of ¾ in. from the surface and which later is seasoned either before erection or while in the structure, 80 per cent of the tabulated loads should be used.

For lumber partially seasoned when fabricated, proportional intermediate values may be used.

For connectors in lumber which is fabricated in a seasoned or unseasoned condition and which will remain wet in service, 67 per cent of the tabulated loads should be used.

111. Connector Units. The allowable connector loads for split rings are based on one ring with its bolt in single shear.

When a joint consists of two connectors used in opposite faces of the timber with the connectors concentric with the same bolt axis or when two or more bolts are used with connectors on separate bolts, the total allowable load is the sum of the allowable loads given for each connector unit. However, if two different sizes of split rings are concentric in the same timber surface, the allowable load is that given for the larger ring only.

112. Edge Distance. Edge distance is the distance from the edge of a member to the center of the connector closest to the edge of the member, measured perpendicular to the grain.

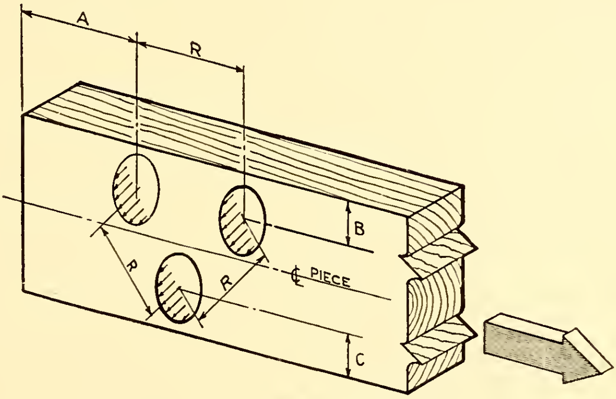

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FIG. 56. Edge and end distance and spacing for connectors: *A*, end distance; *R*, center-to-center spacing; *B* and *C*, edge distance.

There are two types of edge distance: the distance on the unloaded edge and the distance on the loaded or compression edge. When the bearing or load is axial, both edges are unloaded, but, when the load acts toward one edge, the distance measured from the edge toward which the load induced by the connector acts is the loaded edge distance.

In Charts 26 and 27 the permissible edge distances are given for 2½- and 4-in. split rings. For unloaded edge distance the standard and minimum value is that given at the right-hand edge of the charts. For loaded edge distance the value varies in accordance with the angle of load to grain. This value may be determined from Charts 26 and 27 by selecting the line representing the angle of load to grain and the percentage of the full allowable load used. Straight-line interpolation may be used for values intermediate of those given.

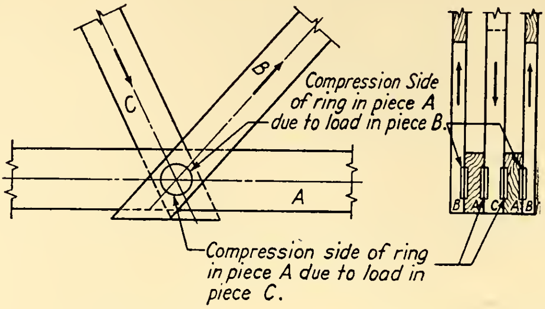
B
C
A
Compression Side of ring in piece A due to load in piece B.
B A C A B
Compression side of ring in piece A due to load in piece C.

Fig. 57. Compression side of ring.

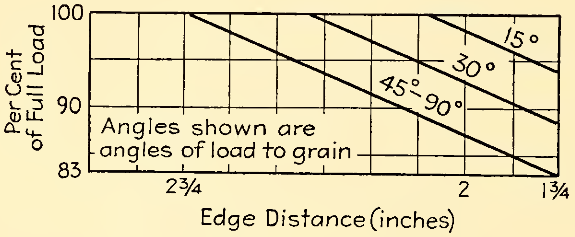

Chart 26. Edge distance for a 2½-in. split ring.

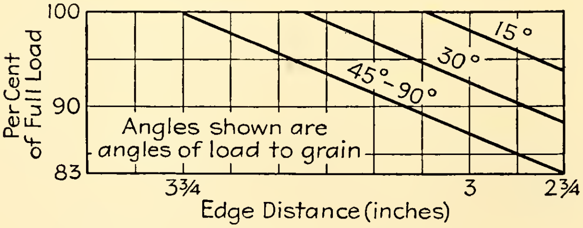

Chart 27. Edge distance for a 4-in. split ring.

113. End Distance. The end distance is measured parallel to the grain from the center of the connector to the square-cut end of the member, as shown in Fig. 56. If the end of the member is not square cut, the end distance should be taken as the distance measured parallel to the centerline of the piece from any point on the center half of the connector diameter which is perpendicular to the centerline of the member. This is illustrated in Fig. 58.

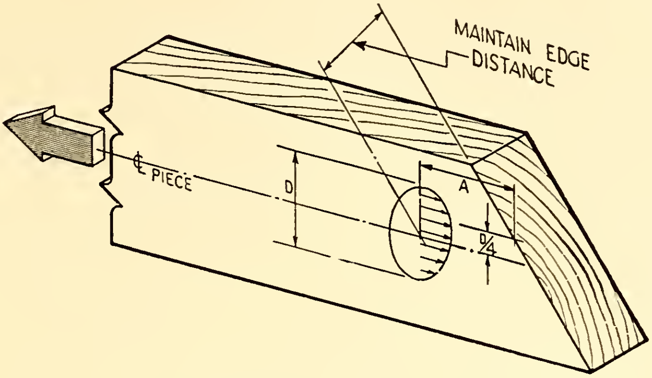

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FIG. 58. End and edge distance for member with sloping end cut.

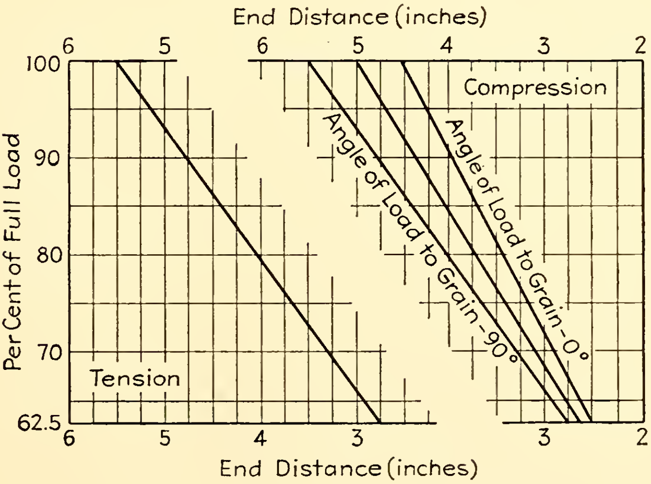

CHART 28. End distance for a 2½-in. split ring.

In Charts 28 and 29 the permissible end distances for split rings are given for both tension and compression members. The end distances for developing 100 per cent of the connector loads are given at the top of the charts at the point coinciding with the slanting line, and the minimum values, or those developing 62.5 per cent of the connector's capacity, are given at the bottom of the charts.

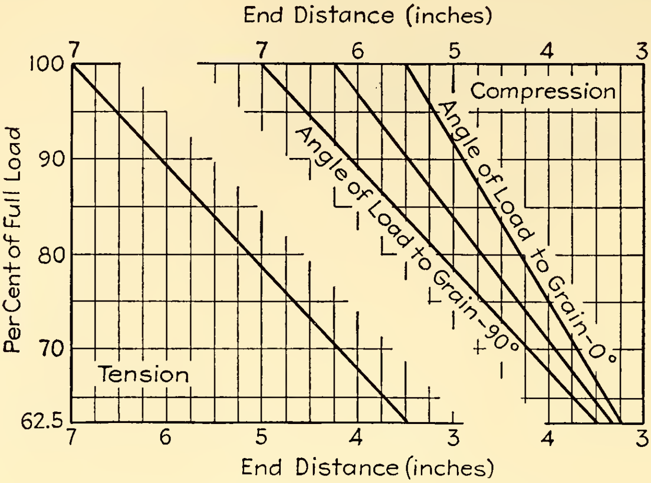

Chart 29. End distance for a 4-in. split ring.

114. Spacing. Spacing is the distance between connectors measured along a line joining their centers as shown in Fig. 56. The center-to-center spacing *R* of pairs of connectors is determined by the intersection of a diameter of an ellipse with its perimeter. The values for the spacing of split rings are determined by placing the coordinates of the ellipse so that one is parallel and the other is perpendicular to the axis of the loaded member. The spacing of the connectors varies as the angle of load to grain varies, and therefore the length of the coordinates of the ellipse will also vary, as illustrated in Fig. 59.

The arrangement of groups of connectors is determined by the minimum spacing permitted. When the connectors are placed about an axis, symmetrical to the direction of the load, the spacing between one pair will often be in excess of that required. In multiple groups of connectors in both faces of a member, the side requiring the greatest spacing governs the location of the connectors.

In Charts 30 and 31 five curves are shown recommending spacing for full load at the particular angle of load to grain as noted on each curve. Straight-line interpolation may be used for intermediate values. The radial lines represent the angle made by the connector axis and the grain. If the spacing for full load is desired, select the proper angle of load to grain curve, and the distance from the point where it intersects the radial line, or the angle made by the connector axis to the grain, to the lower left-hand corner is the minimum spacing required. In detailing the joint it will probably be more convenient to use the parallel and perpendicular to grain components of this spacing.

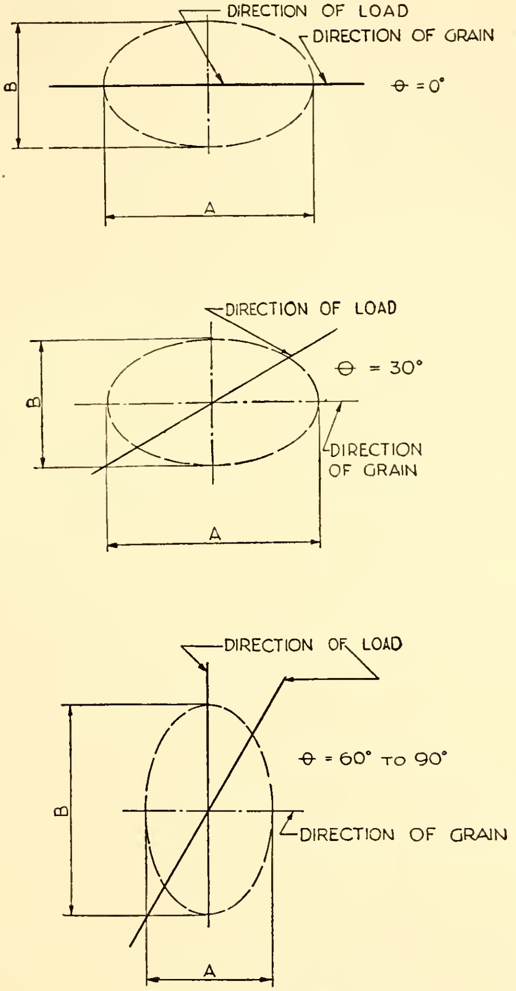
DIRECTION OF LOAD
DIRECTION OF GRAIN
θ = 0°
B
A
DIRECTION OF LOAD
θ = 30°
DIRECTION OF GRAIN
B
A
DIRECTION OF LOAD
θ = 60° TO 90°
DIRECTION OF GRAIN
B
A

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FIG. 59. Variation in coordinates A and B with changes in Θ.

The curve representing 75 per cent of the full load is for any angle of load to grain and is the minimum spacing permitted. For percentages between 75 and 100 per cent, interpolate radially between the 75 per cent curve and the curve representing the curve corresponding to the proper angle of load to grain.

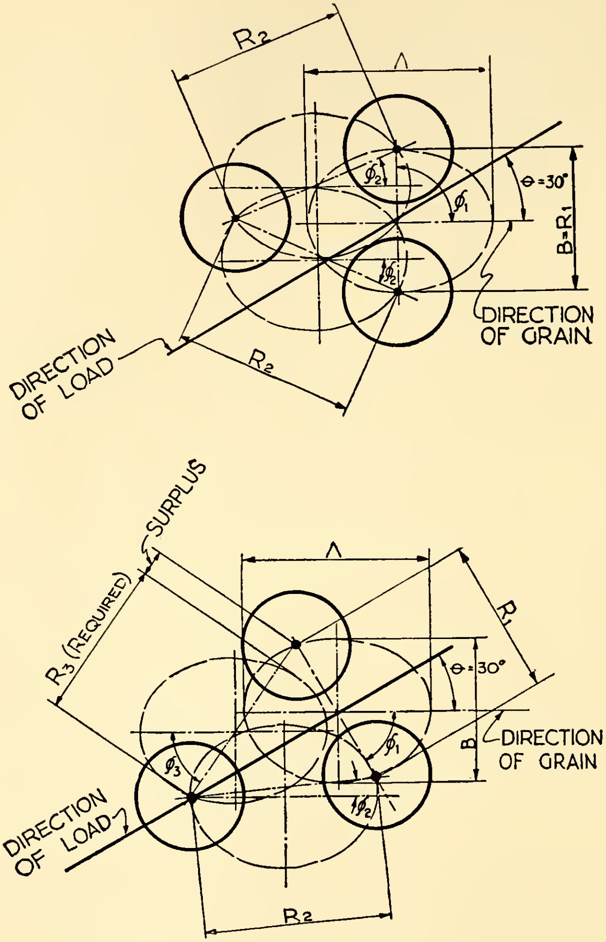
R2
A
θ=30°
φ2
φ1
φ2
B=R1
DIRECTION OF GRAIN
DIRECTION OF LOAD
R2
SURPLUS
R3 (REQUIRED)
A
R1
θ=30°
DIRECTION OF GRAIN
B
φ1
φ3
φ2
DIRECTION OF LOAD
R2

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Fig. 60. Locating three centers.

When a number of connectors are used in the same face of a member and they are not staggered so that the distance measured perpendicular to the grain, between lines parallel to the grain and passing through the center of the connectors, is equal to or greater than the diameter of the connector, the total allowable load for the group of connectors should not exceed the load permitted for one connector multiplied by the percentages in Table 29.

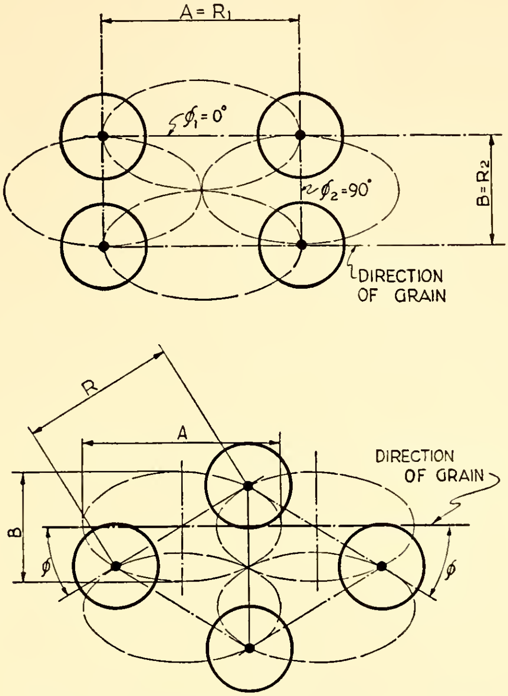

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Fig. 61. Locating four centers.

Table 29. Percentage Values for Use with Multiple Connectors in a Single Piece When Connectors Are Spaced Parallel to the Grain and the Member Is Loaded at Any Angle Between 45° and 90°

Type and Size of Connector	Thickness of Loaded Member (Inches)			
	$1\frac{5}{8}$ in.	2 in.	$2\frac{5}{8}$ in.	3 in. and thicker
$2\frac{1}{2}$-in. split ring	300%	300%	300%	300%
4-in. split ring	233%	248%	269%	300%

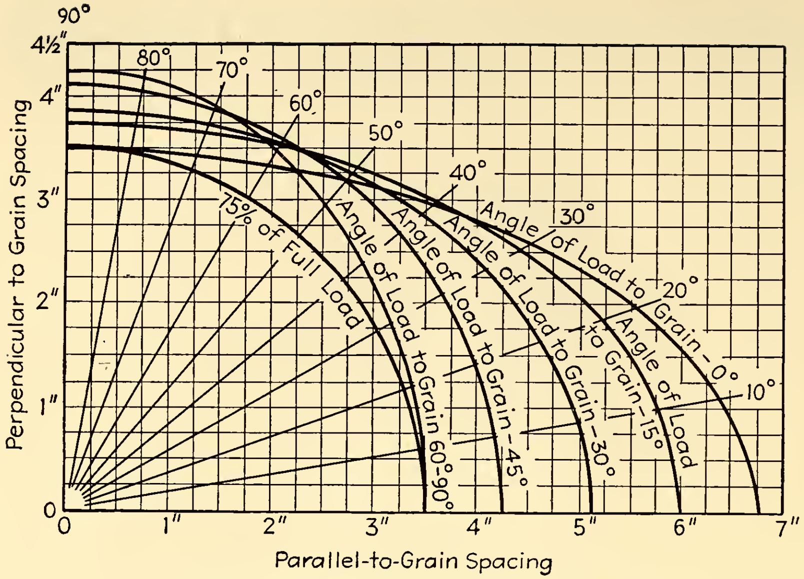


CHART 30. Spacing for a 2½-in. split ring.

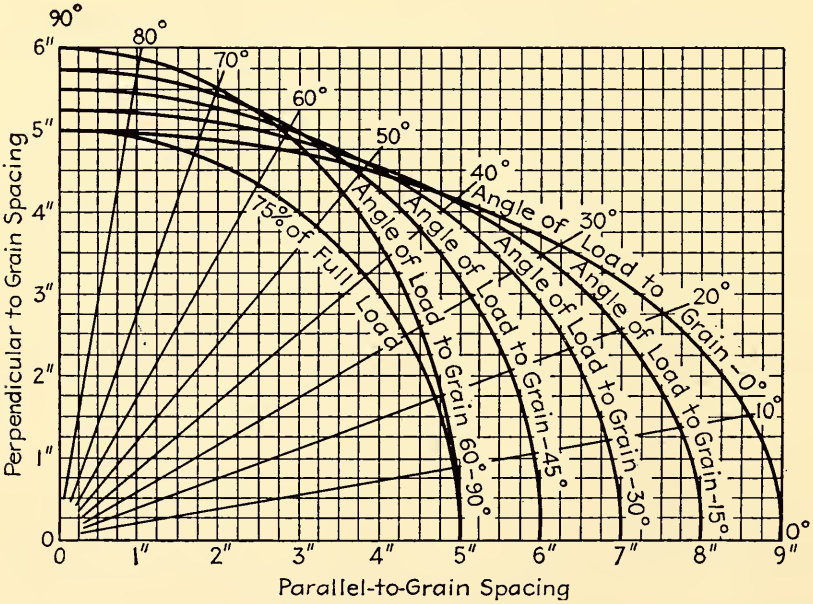


CHART 31. Spacing for a 4-in. split ring.

For conditions existing as set forth for multiple joints classified in Table 29, the spacing should not be less than that for parallel-to-grain spacing at the appropriate angle of load to grain.

115. Interrelationship of Thickness, Distance, and Spacing. When the minimum thicknesses of lumber, as shown on Charts 24 and 25, are used, the allowable connector loads are reduced from a maximum at the standard thickness, and no additional reduction in the capacity of the joint is permissible. This means that no reduction of edge distance, end distance, or spacing is possible.

When the allowable load is reduced because of reduced edge distance, end distance, or spacing, the reduced allowable load for each should be determined separately, and the lowest allowable load so determined for any one connector should apply. Such load reductions are not cumulative. Conversely, if the allowable load is reduced because of a reduced distance or spacing, the other distances or spacings may be reduced; the same reduced allowable load will result.

116. Net Section. The net section of a joint which uses connectors is determined by subtracting from the full cross-sectional area of the member the projected area of that portion of the connectors within the member and that portion of the bolt not within the connector.

Tests have shown that the concentration of stresses at the net section causes failure in tension at stresses approximately equal to the maximum compressive stress of the material parallel to the grain. In terms of safe stresses this value corresponds to the allowable stress in compression parallel to the grain for clear material. On this basis, constants have been determined that can be multiplied by the total load to give the required net section in square inches.

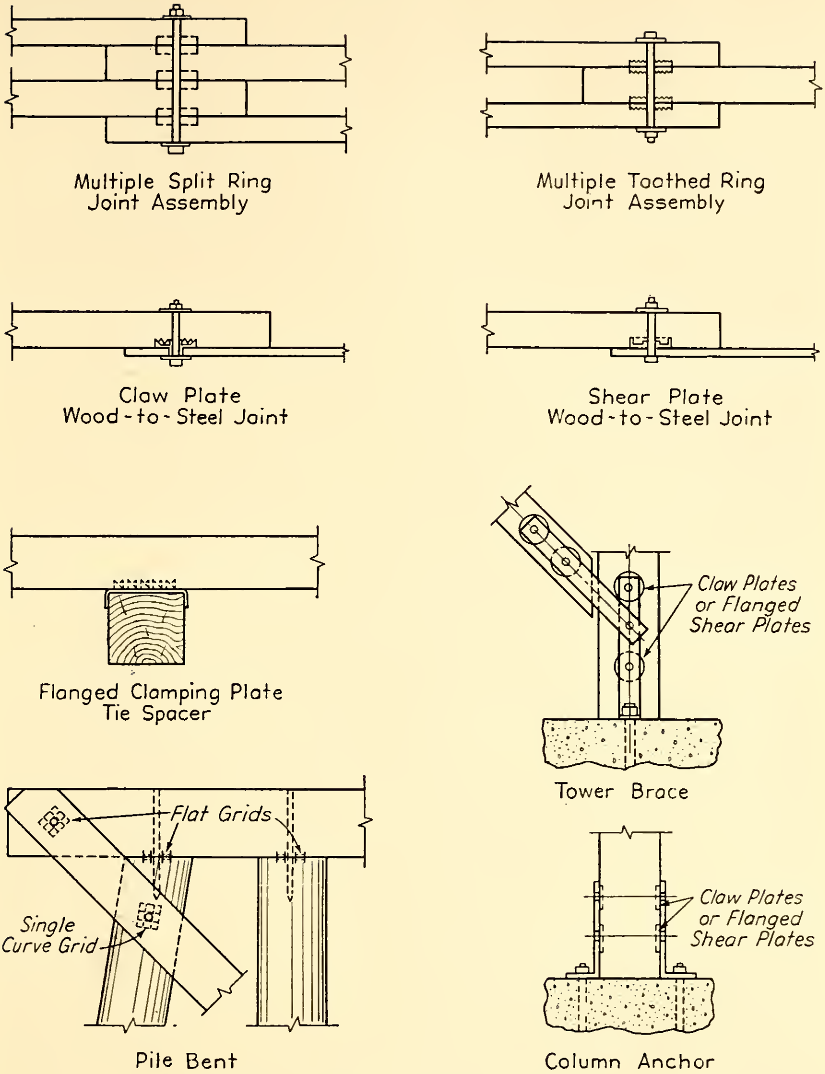
Table 30. Constants for Determining Net Section

Type of Loading	Thickness of Wood Member	Constants for Each Connector Load Group		
		Group A	Group B	Group C
Permanent	4 in. or less	0.00040	0.00047	0.00055
	Over 4 in.	0.00050	0.00059	0.00070
Snow	4 in. or less	0.00035	0.00041	0.00049
	Over 4 in.	0.00044	0.00051	0.00061
Wind or earthquake	4 in. or less	0.00027	0.00031	0.00037
	Over 4 in.	0.00033	0.00039	0.00047

Table 31. Projected Areas of Connectors and Bolts

[Manual of Timber Connector Construction, Timber Engineering Co., 1939.]

Connector		Bolt Diam. (inches)	Placement of Connectors	Total Projected Area in Square Inches of Connectors and Bolts in Lumber Thickness of									
No.	Size			1⅝″	2″	2⅝″	3″	3⅝″	4″	5½″	6″	7½″	8″
Split Rings													
1	2½	½	One Face	1.73	1.92	2.23	2.42	2.73	2.92	3.67	3.92	4.67	4.92
		½	Two Faces	2.64	2.83	3.14	3.34	3.64	3.83	4.58	4.83	5.58	5.83
2	4	¾	One Face	3.09	3.37	3.84	4.12	4.59	4.87	6.00	6.37	7.50	7.87
		¾	Two Faces	4.97	5.25	5.72	6.00	6.47	6.75	7.16	8.25	9.38	9.75
3	6	¾	One Face	4.91	5.19	5.66	5.94	6.41	6.69	7.10	8.19	9.32	9.69
		¾	Two Faces	......	8.88	9.35	9.63	10.10	10.38	10.79	11.88	13.01	13.38
Toothed Rings													
1	2	½	One Face	1.52	1.71	2.02	2.21	2.52	2.71	3.46	3.71	4.46	4.71
		½	Two Faces	2.22	2.41	2.72	2.91	3.22	3.41	4.16	4.41	5.16	5.41
2	2⅝	⅝	One Face	1.95	2.18	2.58	2.81	3.20	3.44	4.37	4.69	5.62	5.94
		⅝	Two Faces	2.89	3.12	3.52	3.75	4.14	4.38	5.31	5.63	6.56	6.88
3	3⅜	¾	One Face	2.46	2.74	3.21	3.49	3.96	4.24	5.37	5.74	6.87	7.24
		¾	Two Faces	3.70	3.98	4.45	4.73	5.20	5.48	6.61	6.98	8.11	8.48
4	4	¾	One Face	2.75	3.03	3.50	3.78	4.25	4.53	5.66	6.03	7.16	7.53
		¾	Two Faces	4.28	4.56	5.03	5.31	5.78	6.06	7.19	7.56	8.69	9.06
Claw Plates													
1 and 1A	2⅝	½	One Face	2.41	2.60	2.91	3.10	3.41	3.60	4.35	4.60	5.53	5.69
		½	Two Faces	......	4.19	4.50	4.69	5.00	5.19	5.94	6.19	6.94	7.19
2 and 2A	3⅛	½	One Face	2.78	2.97	3.28	3.47	3.78	3.97	4.72	4.97	5.72	5.97
		½	Two Faces	......	4.93	5.24	5.43	5.74	5.93	6.68	6.93	7.68	7.93
3 and 3A	4	¾	One Face	3.66	3.94	4.41	4.69	5.16	5.44	6.57	6.94	8.07	8.44
		¾	Two Faces	......	6.37	6.84	7.12	7.59	7.87	9.00	9.37	10.50	10.87
Shear Plates													
1	2⅝	¾	One Face	1.92	2.20	2.67	2.95	3.42	3.70	4.83	5.20	6.33	6.70
		¾	Two Faces	2.62	2.90	3.37	3.65	4.12	4.40	5.53	5.90	7.03	7.40
2	4	¾	One Face	3.23	3.51	3.98	4.26	4.73	5.01	6.14	6.51	7.64	8.01
		¾	Two Faces	......	5.52	5.99	6.27	6.74	7.02	8.15	8.52	9.65	10.02
2-A	4	⅞	One Face	3.36	3.69	4.24	4.56	5.11	5.44	6.75	7.19	8.50	8.94
		⅞	Two Faces	......	5.62	6.17	6.49	7.04	7.37	8.68	8.12	10.43	10.87
Spike Grids													
1, 2 and 3	4⅛ × 4⅛	¾	One Face	2.90	3.18	3.65	3.93	4.40	4.68	5.81	6.18	7.31	7.68
		¾	Two Faces	......	4.87	5.34	5.62	6.09	6.37	7.50	7.87	9.00	9.37
		1	One Face	3.18	3.56	4.18	4.56	5.18	5.56	7.06	7.56	9.06	9.56
		1	Two Faces	......	5.13	5.75	6.13	6.75	7.13	8.63	9.13	10.63	11.13


Multiple Split Ring Joint Assembly

Multiple Toothed Ring Joint Assembly

Claw Plate Wood-to-Steel Joint

Shear Plate Wood-to-Steel Joint

Flanged Clamping Plate Tie Spacer

Tower Brace

Pile Bent

Column Anchor

Claw Plates or Flanged Shear Plates
Flat Grids
Single Curve Grid
Claw Plates or Flanged Shear Plates

FIG. 62. Typical joints with connectors.

117. Fabrication. The split rings, shear plates, and claw plates are installed into grooves or daps that are cut by means of special tools. The Timber Engineering Company manufactures two types: one with a circular cutter head with replaceable knives and a pilot, and the other with a two-wing cutter head with knives attached and a pilot.

Claw plates fit partly into daps, and the teeth are embedded in the wood by driving or by pressure. The toothed rings and spike grids are installed by pressure by means of a high strength rod and a ball-bearing washer.

The high strength rod should have the same diameter as the bolt to be used in the joint and should be used with plate washers whose diameters equal the diameters of the connectors.

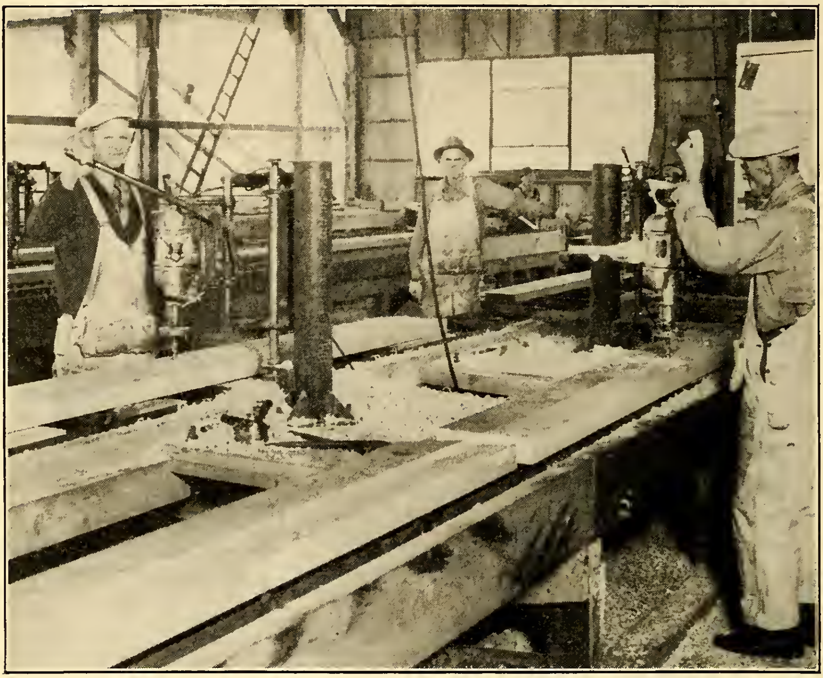


Fig. 63. Fabricating truss members in shop of Timber Structures, Inc., Portland, Oregon.

Flat clamping plates are embedded in the wood by pressure or by driving. Usually a driving ram is used in combination with a heavy steel seating block with slots to protect the teeth. Flanged clamping plates have teeth on one side only and are driven into the guard timber by means of a heavy ram and a cover plate to protect the plate against bending.

When several identical trusses or arches are to be used in a structure, it is best to prepare templates for the individual members. These should have the bolt holes in them and should be cut to the exact shape of the member. Additional markings can be used on the templates for locating grooves or for identification purposes.

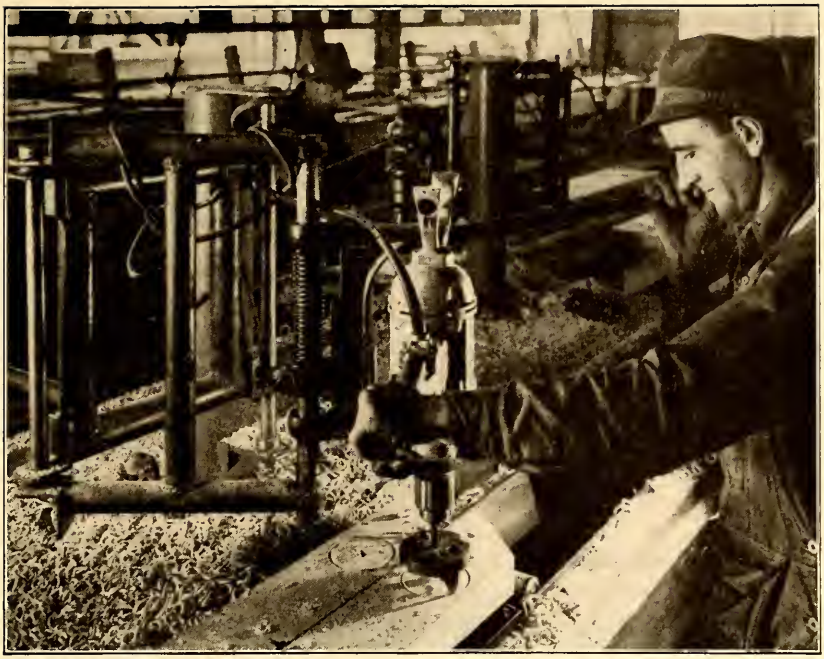


FIG. 64. Cutting grooves for split rings in shop of Timber Structures, Inc.

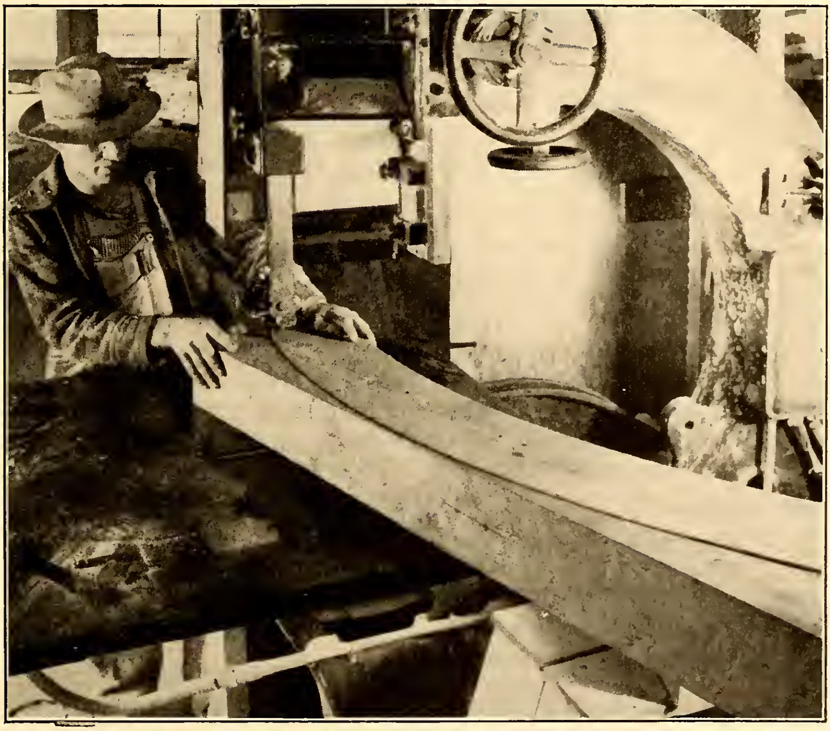


FIG. 65. Fabrication of curved chord members.

ILLUSTRATIVE PROBLEMS

Problem. Find the number and size of split-ring connectors to carry 25,000 lb. in a lap tension joint. Use a No. 1 dense grade of southern pine (1700 f).

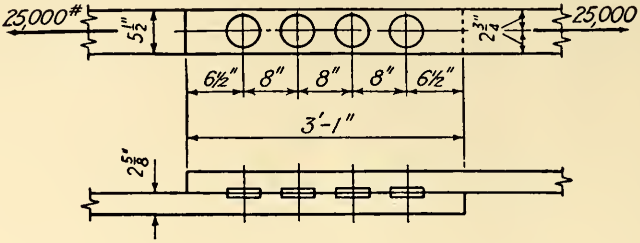


Fig. 66. Lap-tension joint using split rings.

$$\text{Required area} = \frac{25{,}000}{1700} = 14.70 \text{ sq. in.}$$

A 3 by 6 in. member has an area of 14.77 sq. in.

A 2 by 10 in. member has an area of 15.44 sq. in.

Either two rows of 2½-in. rings or one row of 4-in. rings can be used in the 2 by 10 in. member or one row of 4-in. rings can be used in the 3 by 6 in. member.

Number of split rings required. From Chart 24, the safe load for 2½-in. rings in one face of 1⅝-in. lumber = 3450 lb.

$$\text{Number of rings} = \frac{25{,}000}{3450} = 7.25$$

The safe load for 4-in. rings in one face of 1⅝-in. lumber = 6690 lb.

$$\text{Number of 4-in. rings} = \frac{25{,}000}{6690} = 3.74 \quad \text{(use 4)}$$

This same number is required in 2⅝-in. lumber.

It is best to keep the number of bolts at a minimum, and the 4-in. rings with 3 by 6 in. material are recommended.

Net section:

From Table 30, the constant 0.00040 is selected.
Required net section = 25,000 × 0.00040 = 10.00 sq. in.
From Table 31, the projected area of a 4-in. ring and bolt in lumber 2⅝ in. thick = 3.84 sq. in.
Actual net section = 14.77 − 3.84 = 10.93 sq. in.

Reduced spacing:

Capacity of rings = 4 × 6690 = 26,800 lb.

$$\text{Percentage of capacity developed} = \frac{25{,}000}{26{,}800} = 93.25 \text{ per cent.}$$

Reduced spacing = 8 in.
Reduced end distance = 6½ in.

Problem. Design and detail a tension joint, using split-ring connectors to carry 55,000 lb. Use No. 1 dense southern pine (1700 f).

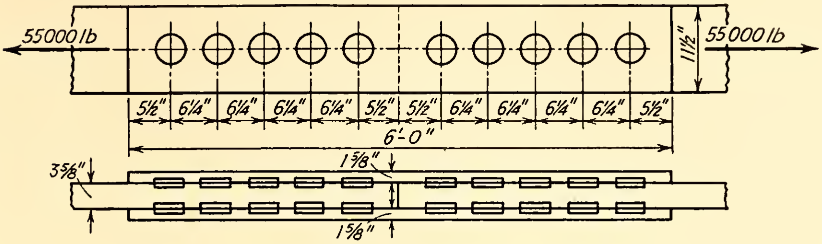


FIG. 67. Split-ring tension joint.

$$\text{Size of members: } \frac{55{,}000}{1700} = 32.35 \text{ sq. in.}$$

Use 4 by 12 in., area = 41.69 sq. in.
Use 2 by 12 in. splice plates.

$$\text{Number of 4-in. rings} = \frac{55{,}000}{6690} = 8.22 \quad \text{(use 10)}$$

Spacing parallel to grain: standard = 9 in., minimum = 5 in.
End distance: standard = 7 in., minimum = 3½ in.
Edge distance: standard and minimum = 2¾ in.
Net area: required = 55,000 × 0.00050 = 27.5 sq. in.
Actual = 41.69 − 6.47 = 35.22 sq. in.

Reduced Spacing:

$$\text{Percentage of capacity developed} = \frac{55{,}000}{10 \times 6690} = 82.2 \text{ per cent.}$$

Reduced spacing = 6¼ in.
End distance = 5½ in.

Remarks. A comparison of this joint with the one previously designed for bolts only will show the advantages of connectors.

118. Multiple Joints. Because the load-carrying capacity of connectors decreases as the angle between the direction of the applied load and the grain of the wood increases, it is important to know which member in a joint is being acted upon at an angle to the grain. In the heel joint of a truss the vertical component of the load transmitted by the top chord is usually counteracted by the reaction of the supporting column of the truss. This leaves the horizontal component, which is the stress in the bottom chord, and the load on the connectors in this member is parallel to the grain. However, an equal and opposite load which acts on the connectors in the top chord is acting at an angle to the grain

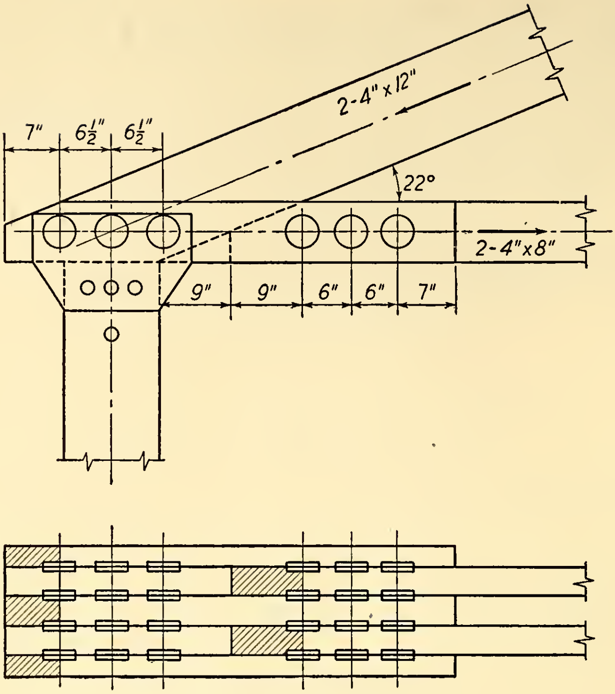

7"
6½"
6½"
2-4" x 12"
22°
2-4"x8"
9"
9"
6"
6"
7"

Fig. 68. Typical heel joint.

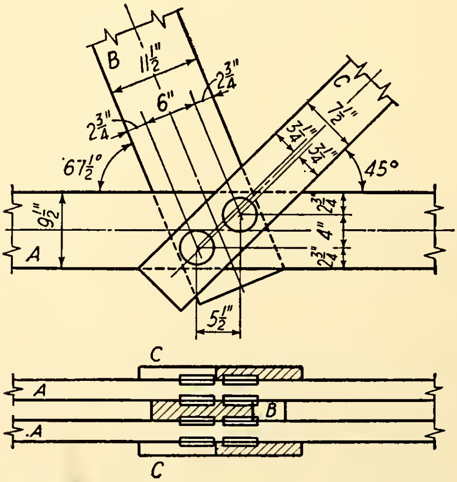

B
11½"
2¾"
6"
2¾"
67½°
C
7½"
3¼"
3¼"
45°
9½"
A
2¾"
4"
2¾"
5½"
C
A
B
A
C

Fig. 69. Typical chord joint in a Fink truss.

in this member equal to the angle between the two members. The number of connectors in the joint is then determined by their load capacities at this angle.

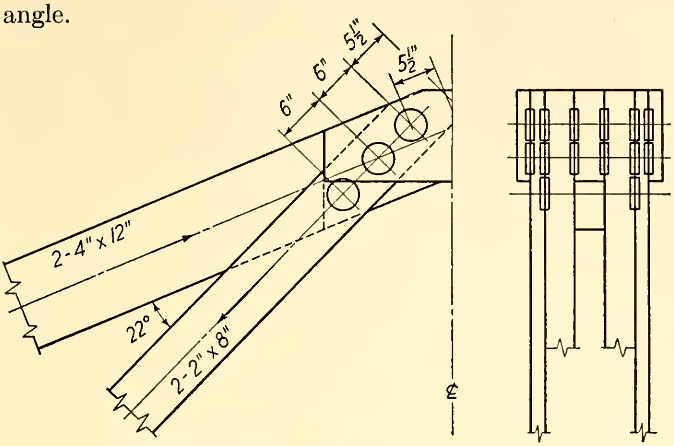


Fig. 70. Typical peak joint in a Fink truss.

For a joint where two web or diagonal members enter on each side of a horizontal member, the vertical reactions are usually equal and opposite in direction, and the joint is designed with load capacities of the connectors determined by the angle of load to grain that each side member makes with the center member. In a three-member joint it is generally the center member in which the load acts at an angle to the grain; in a five-member joint it is usually the second and fourth members that are loaded at an angle to the grain.

When two or more members form a joint and the angle of load to grain is less than 30° for any two contacting members, the connectors can be spaced with reference to any member. This situation occurs in the peak joint of a Fink truss.

When the members enter a joint such as is found in a flat-top Pratt truss and the angle between the members is greater than 30°, the spacing of the connectors must be considered in the member that has the load applied at the greatest angle.

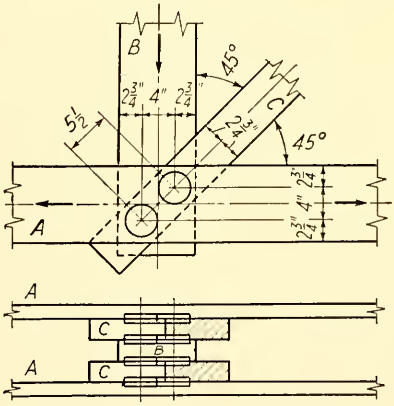


Fig. 71. Typical joint in a Pratt truss.

Design Example. Consider the joints shown in Figs. 72 and 73, and determine the size of the members and the number of connectors to satisfy the conditions of the joint. As shown in the figures, there are two methods of placing the connectors in the joint. They can be spaced either parallel or perpendicular to the grain in member No. 2.

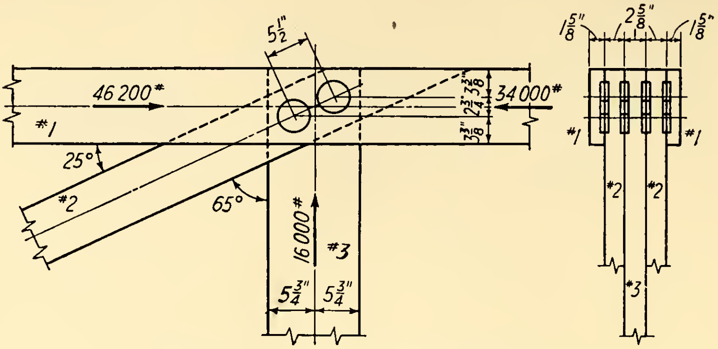


FIG. 72. Five-member split-ring joint.

Split-Ring Connector Data	*Vertical to Diagonals*	*Chords to Diagonals*
Size	4 in.	4 in.
Number of faces	2	2
Thickness of piece	$2\frac{5}{8}$ in.	$2\frac{5}{8}$ in.
Angle	65°	25°
Standard design load	4,820 lb.	6,100 lb.
Joint stress	16,000 lb.	12,200 lb.
Connectors required	$\frac{16,000}{4,820} = 3.32$	$\frac{12,200}{6,100} = 2.00$
Connectors used	4	4
Capacity developed	$\frac{3.32}{4} = 83\%$	$\frac{2}{4} = 50\%$
Spacing required	5 in.	$7\frac{1}{2}$ in. to 5 in.
Spacing used	$5\frac{1}{2}$ in.	$5\frac{1}{2}$ in.
Edge distance:		
Compression side diagonal	$2\frac{3}{4}$ in.	$2\frac{3}{4}$ in.
Vertical	$2\frac{3}{4}$ in.	$2\frac{3}{4}$ in.
Chord	$2\frac{3}{4}$ in.	$2\frac{3}{4}$ in.
Width of diagonal	$2\frac{3}{4} + 2\frac{3}{4} = 5\frac{1}{2}$ in.	
Width of vertical	$2\frac{3}{4} + 5 + 2\frac{3}{4} = 10\frac{1}{2}$ in.	
Width of chord	$2\frac{3}{4} + 2.75 + 2\frac{3}{4} = 8.25$ in.	
Size of members:		
No. 1 chords	2—2 in. by 10 in.	
No. 2 diagonals	2—3 in. by 6 in.	
No. 3 vertical	1—3 in. by 12 in.	

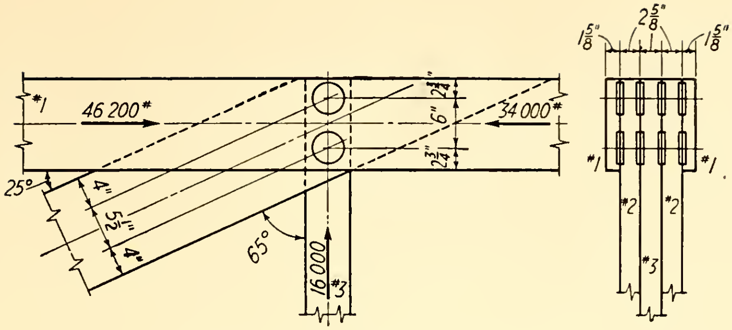


FIG. 73. Five-member split-ring joint.

Split-Ring Connector Data	*Vertical to Diagonals*	*Chords to Diagonals*
Size	4 in.	4 in.
Number of faces	2	2
Thickness of piece	$2\frac{5}{8}$ in.	$2\frac{5}{8}$ in.
Angle	65°	25°
Standard design load	4,820 lb.	6,100 lb.
Joint stress	16,000 lb.	12,200 lb.
Connectors required	$\frac{16,000}{4,820} = 3.32$	$\frac{12,200}{6,100} = 2.00$
Connectors used	4	4
Capacity developed	$\frac{3.32}{4} = 83\%$	$\frac{2}{4} = 50\%$
Spacing required	$5\frac{1}{4}$ in.	$5\frac{1}{2}$ in.
Spacing used	$5\frac{1}{2}$ in.	$5\frac{1}{2}$ in.
Edge distance:		
Compression side diagonal	$3\frac{1}{2}$ in.	$3\frac{1}{2}$ in.
Chord	$2\frac{3}{4}$ in.	$2\frac{3}{4}$ in.
Vertical	$2\frac{3}{4}$ in.	$2\frac{3}{4}$ in.

Width of diagonal	$3\frac{1}{2} + 5\frac{1}{2} + 3\frac{1}{2} = 12\frac{1}{2}$ in.
Width of chord	$2\frac{3}{4} + 6 + 2\frac{3}{4} = 11\frac{1}{2}$ in.
Width of vertical	$2\frac{3}{4} + 2\frac{3}{4} = 5\frac{1}{2}$ in.
Size of members:	
No. 1 chords	2—2 in. by 12 in.
No. 2 diagonals	2—3 in. by 14 in.
No. 3 vertical	1—3 in. by 6 in.

In the previous examples the end distances were not computed. The end distance for either tension or compression members with connectors bearing axially is determined by calculating the required percentage by multiplying the percentage of the basic design load for the angle of load to grain in the loaded member and the used connector capacity percentage. In a loaded tension member with connectors on both sides bearing in opposite directions, the end distance is constant for all angles of load to grain, but in a similarly loaded compression member the end distance varies with the angle of load to grain.

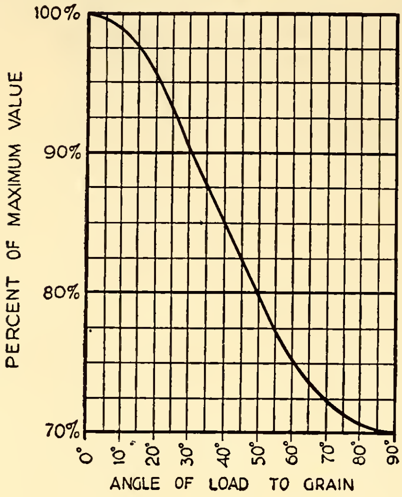


Chart 32. Percentage of basic load for split rings when the load is applied at any angle to the grain.

For convenience in determining the percentage of the basic load, or maximum load, for split rings, when the load is applied at any angle to the grain, Chart 32 may be used.

In the following example the method of determining end distances and locating a group of connectors is illustrated.

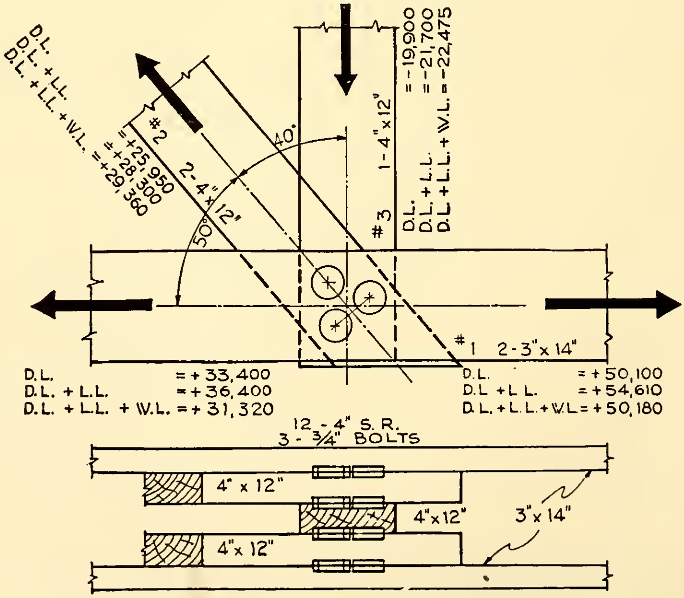

D.L. = +25,950
D.L. + L.L. = +28,300
D.L. + L.L. + W.L. = +29,360
#2
2-4"x12"
40°
50°
#3
1-4"x12"
D.L. = -19,900
D.L. + L.L. = -21,700
D.L. + L.L. + W.L. = -22,475
#1
2-3"x 14"
D.L. = +33,400
D.L. + L.L. = +36,400
D.L. + L.L. + W.L. = +31,320
D.L. = +50,100
D.L + L.L. = +54,610
D.L.+L.L.+W.L = +50,180
12 - 4" S. R.
3 - 3/4" BOLTS
4" x 12"
4"x12"
4"x 12"
3"x 14"

Fig. 74. Lower chord joint in a Pratt truss.

Connector Data and Loads	*Vertical to Diagonals*	*Chords to Diagonals*
Size and type	4-in. split ring	4-in. split ring
Number of faces	2	2
Thickness of piece	$3\frac{5}{8}$ in.	$3\frac{5}{8}$ in.
Species group	B	B
Moisture content condition	-20%	-20%
Basic design load	4,590	4,590
Angle of load to grain θ	40°	50°
Percentage of basic load based on θ	85%	80%
Connector capacity at angle θ		
Permanent loading L	3,900	3,670
Three months loading, $L \times 1.15 = L_1$	4,490	4,220
Wind or earthquake loading, $L \times 1.33 = L_2$	5,190	4,880
Joint stress		
Dead load P	19,900	16,700
Dead load + live load, P_1	21,700	18,210
Dead load + live load + W. load, P_2	22,475	18,860
Connectors required		
P/L	5.10	4.55
P_1/L_1	4.84	4.32
P_2/L_2	4.33	3.86
Connectors used	6	6
Capacity developed	$\frac{5.10}{6} = 85\%$	$\frac{4.55}{6} = 76\%$
Spacing required[1]		
ϕ	90° 30°	90° 30°
R	$5\frac{3}{8}$ and $5\frac{1}{2}$	$5\frac{1}{8}$ and $5\frac{1}{8}$
Spacing used[1]	$5\frac{1}{2}$ in.	$5\frac{1}{2}$ in.
Edge distance		
Compression side diagonal	$2\frac{3}{4}$ in.	$2\frac{3}{4}$ in.
Vertical	$2\frac{3}{4}$ in.	$2\frac{3}{4}$ in.
Chord	$2\frac{3}{4}$ in.	$2\frac{3}{4}$ in.
End distance		
Diagonal, 85%	$5\frac{5}{8}$ in.	
Vertical, $0.85 \times 0.85 = 72\frac{1}{4}\%$	$3\frac{7}{8}$ in.	
Width of diagonal	$2\frac{3}{4} + 5\frac{1}{2} + 2\frac{3}{4} = 11$ in.	

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[1] See footnote on next page.

Connector Data and Loads	*Vertical to Diagonals*	*Chords to Diagonals*
Width of vertical	$2\frac{3}{4} + 5 + 2\frac{3}{4} = 10\frac{1}{2}$ in.	
Width of chord	$2\frac{3}{4} + 5\frac{1}{2} + 2\frac{3}{4} = 11$ in. (use $13\frac{1}{2}$ to carry stress)	
Size of members		
No. 1 chords	2—3 in. by 14 in.	
No. 2 diagonals	2—4 in. by 12 in.	
No. 3 vertical	1—4 in. by 12 in.	

[1] The calculations for the center-to-center spacing may be divided into two steps:

1. Location of two of the three centers involved.

Known

Angle of load to grain, θ = 40°
Angle of axis of connectors, ϕ = 90°
Percentage of full connector load = 85%
R from Chart 31 = $5\frac{3}{8}$ in.
R used = $5\frac{1}{2}$ in

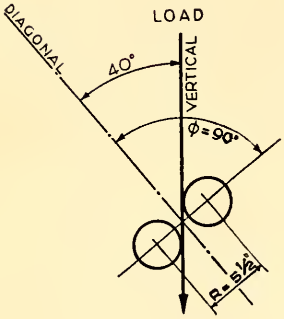
DIAGONAL
LOAD
VERTICAL
40°
φ=90°
R=5½"

FIG. 75. Location of two centers.

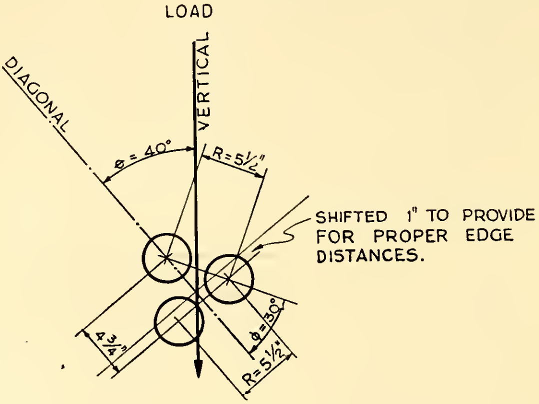
LOAD
VERTICAL
DIAGONAL
φ = 40°
R=5½"
SHIFTED 1" TO PROVIDE FOR PROPER EDGE DISTANCES.
4¾"
φ=30°
R=5½"

FIG. 76. Location of third center.

2. Location of two of the three centers by including either center located in step No. 1.

Known

Angle of load to grain, θ = 40°

Assume

Angle of axis of connectors, ϕ = 30°

Solve for

R from Chart 31 = $5\frac{1}{2}$ in.

By considering the load in the chord transferred to the diagonal and calculating R in the same manner it will be found that the value for R is less. Since the load in the vertical member requires the greater spacing, this value of R will govern the design.

119. Eccentric or Moment-Resisting Joints. A series of tests recently conducted by the Timber Engineering Company for the purpose of developing an empirical formula or method of design for split-ring joints that have eccentricity has resulted in a recommended procedure:

1. Design the members and the connections as though the joint were concentric.

2. Place the connectors in the critical piece so that the maximum possible effective depth d_e is realized. The effective depth is the distance from the inside edge of the split ring nearest the unloaded edge of the member to the loaded edge of the member.

3. Check the critical member for the fiber stress developed due to bending and direct stress or due to bending only if direct stress does not exist.

4. Check the critical member for horizontal shear using the effective depth in the usual shear formula, and use an increase of 50 per cent over the allowable horizontal shear value so that

$$1.5q = \frac{3V}{2bd_e}$$

where q = allowable unit stress in horizontal shear in pounds per square inch.
V = vertical shear in pounds.
b = width of section in inches.
d_e = effective depth in inches.

5. If the horizontal shear values are excessive, change the joint detail to increase the effective depth or effective area.

PROBLEMS

1. Find the safe working stress when a load is applied at an angle of 60° in a dense No. 1 structural grade of southern pine.

2. Calculate the safe working stress for a 1700 f * grade of Douglas fir (coast region) when the load is applied at an angle of 45° to the grain.

3. Determine the number and size of nails and common screws to resist a pull of 3000 lb. in the direction of their length when they are used to fasten a piece 1 in. thick to a piece 6 in. thick. Use Douglas fir.

4. Find the safe load for a tension joint with wood splice plates and six ½-in. bolts on each side of the joint. Use (*a*) southern pine, (*b*) eastern hemlock, and (*c*) tupelo. Assume a main member 4 in. thick.

5. Find the safe load for a bolted joint with six ½-in. bolts when the load is applied at an angle of 45°. Assume a main member 4 in. thick and two side members each 2 in. thick. Use (*a*) Douglas fir (Rocky Mountain region), (*b*) rock elm, and (*c*) red oak.

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* Refers to *The Grade of Lumber.*

6. Find the number and size of spikes, lag screws, and bolts to resist a lateral force of 3000 lb. between a 2-in. and 4-in. piece of seasoned southern pine. (*a*) Consider the load parallel to the grain of both pieces. (*b*) Consider the load perpendicular to the grain of the 4-in. piece.

7. Design and detail a bolted joint to carry 20,000 lb. in tension. Assume that the joint will be exposed to the weather. Use southern pine, and consider the main member 6 in. thick. (*a*) Load applied parallel to grain in main member. (*b*) Load applied perpendicular to grain in main member.

8. Find the number and size of bolts and split-ring connectors with bolts in each member of a two-member joint when a load of 15,000 lb. is applied in tension in one of the members. The members butt at an angle of 45°. Use 1700*f* Douglas fir, and consider both members to be 4 in. thick.

9. Two members 6 in. thick join at an angle of 60°, and one member carries a load of 22,000 lb. in compression. Using metal splice plates and bolts, design and detail the joint. Use southern cypress.

10. In a lap tension joint carrying 30,000 lb., determine the size of the members, the number of split rings required, and detail the joint, taking full advantage of reduction in spacing and end distance. Use dense No. 1 structural southern pine.

CHAPTER VI

BEAMS AND COLUMNS

BEAMS

120. General. In designing wood beams three stresses must be considered. These are the extreme fiber stress in flexure, the maximum horizontal shear stress, and the stress in compression across the grain at the reactions or concentrated loads. Besides these stresses, which must not exceed the allowable value in each case, the beam must possess the stiffness required in the particular use for which it is intended.

121. Extreme Fiber Stress. The size of a beam to resist a bending moment due to a load may be determined by the familiar formula

$$M = \frac{fI}{c}$$

where M = bending moment.
f = allowable fiber stress for the grade of lumber used.
I = moment of inertia of the section.
c = distance from the neutral axis to the extreme fiber.

For practical purposes it is assumed that the neutral axis is located at the center of the beam; however, this is not exactly correct because wood is a nonhomogeneous material.

A wood beam will carry a much greater load for a few minutes than for several years, and for this reason impact stresses that do not exceed the live load stresses may be neglected.

122. Horizontal Shear. When the design of rectangular beams is governed by the allowable unit stress in horizontal shear, the usual formula is

$$q = \frac{3V}{2bd}$$

where q = maximum horizontal shear stress.
V = external shear.
b = width of the beam.
d = depth.

However, this formula should be modified because the grading rules for structural lumber permit certain depths of checks and shakes at the neutral plane. Tests at the Forest Products Laboratory have shown that when these checks or shakes are present, the upper and lower portions of a beam act partly as two beams and partly as a unit. For this reason, part of the end reaction is resisted internally by each half of the beam acting independently and consequently is not associated with shearing stress at the neutral plane.*

The following procedure is recommended by the Forest Products Laboratory for calculating the horizontal shear on the neutral plane in checked beams.

1. Use the ordinary shear formula and the unit stress in horizontal shear for the particular structural grade selected.

2. In calculating the reaction for use in the shear formula (*a*) disregard all loads within the height of the beam from both supports, (*b*) place the heaviest moving load at three times the height of the beam from the support, and (*c*) treat all other loads in the usual manner.

If the beam does not qualify under the foregoing recommendation, the reactions should be determined by the following precise equation:

$$R = \frac{10P(L - x)\left(\frac{x}{h}\right)^2}{9L\left[2 + \left(\frac{x}{h}\right)^2\right]}$$

where R = the reaction due to the load P.
L = the span in inches.
x = the distance in inches from the reaction to the load P.
h = the height of the beam in inches.

If the load is a moving one, the value of x for use in the above formula may be found from the following equation:

$$Z^3 + 6Z = 4U$$

where $Z = x/h$ and $U = L/h$.

123. Bearing. The allowable unit stress in compression perpendicular to the grain should be equal to or greater than the value obtained by dividing the end reaction by the end-bearing area of the beam. The working stresses in compression perpendicular to the grain given in Table 8 apply to bearings of any length at the ends of a beam and to bearings of 6 in. or more at any other point in the length of the beam. If the bearings are shorter than 6 in. and located 3 in. or more from the end of a beam, the working stresses may be increased in accordance with the values given in Table 9.

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* "New Method of Calculating Longitudinal Shear in Wooden Beams," by J. A. Newlin, G. E. Heck, and H. W. March, *Eng. News-Record,* 110: 594–596.

124. Deflection. The dimensions of a beam designed for stiffness may be calculated by means of the usual deflection formulas, and the deflection is generally limited to $1/360$ of the span. For highway bridges a ratio of $1/200$ of the span is commonly used, and for stringers in railroad bridges and trestles limitations range from $1/200$ to $1/300$. Wood beams usually sag in time; consequently the deflection will increase. For this reason it is customary to double the dead load in computing the deflection.

125. Notched Beams. Beams are often notched at the ends to allow more clearance or to bring the top surfaces level with adjacent beams. Notches at intermediate points are not uncommon because it is frequently necessary to allow space for pipes and other parts of a structure.

When beams are notched at the end, as shown in Fig. 77, the bending load must be checked against the load obtained by the formula

$$V = \frac{2}{3}\,\frac{bd_1^2 q}{d}$$

where V = end reaction.
b = width of beam.
d_1 = end depth above the notch.
q = working stress in horizontal shear.
d = total depth of beam.

When notches are located at or near the middle of a beam, the net depth should be used in obtaining the bending strength.

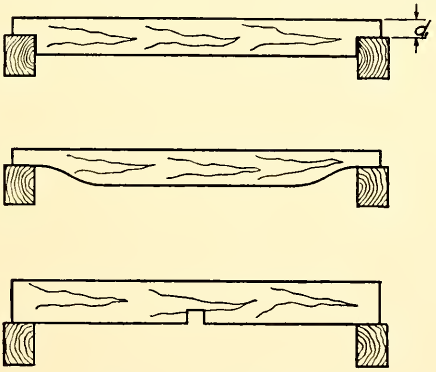


Fig. 77. Notched beams.

126. Buckling. Narrow and deep beams unsupported laterally will often fail in torsion due to lateral buckling. The buckling load for a beam supported and held vertical at its ends, but not otherwise restrained, and carrying a uniformly distributed load may be calculated from the formula

$$W = \frac{28.3\sqrt{EI^1GK}}{L^2}$$

For a beam under the same conditions as above except that in addition to being held vertical it is clamped at the ends to prevent lateral rotation, the formula becomes

$$W = \frac{43.3\sqrt{EI^1GK}}{L^2}$$

where W = buckling load.
E = modulus of elasticity.
I^1 = moment of inertia of beam about its vertical axis.
G = modulus of rigidity in torsion = $E/16$.
L = span in inches.
K = torsion constant of the section and is equal to βdb^3 in which β is a constant depending upon the ratio of the sides d/b.

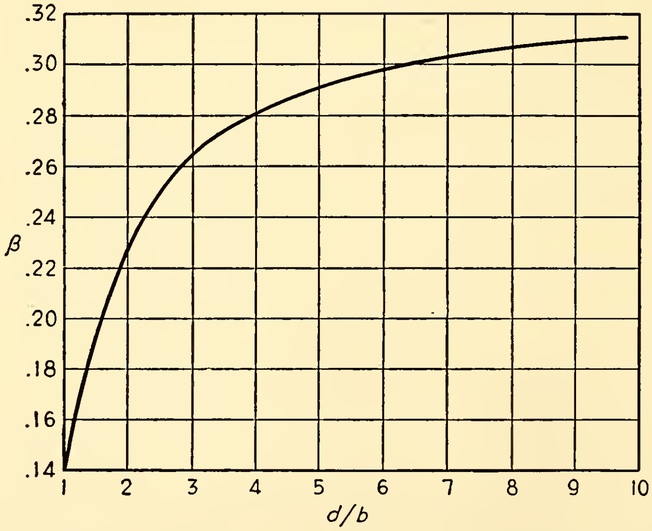


Chart 33. Values of β in terms of the ratio of the sides d/b.

ILLUSTRATIVE PROBLEMS

Problem. Design a beam with a span of 16 ft. 0 in. to carry a load of 1000 lb. per linear foot. The beam is to be of Douglas fir, for which the allowable unit fiber stress is 1450 lb. per sq. in., and the allowable unit horizontal shear is 120 lb. per sq. in. The modulus of elasticity is 1,600,000 lb. per sq. in.

$$M = \frac{wl^2}{8} = \frac{1000 \times 16 \times 16 \times 12}{8} = 384{,}000 \text{ in.-lb.}$$

$$V = \frac{wl}{2} = \frac{1000 \times 16}{2} = 8000 \text{ lb.}$$

$$f = \frac{Mc}{I} = \frac{M6}{bd^2} \qquad bd^2 = \frac{384{,}000 \times 6}{1450} = 1590 \text{ in.}^3$$

Let $b = 9.5$ in. (10 in. nominal).

$$d = \sqrt{\frac{1590}{9.5}} = 12.9 \text{ in.}$$

A beam 10 by 14 in. will satisfy the bending requirements.

$$V^1 = \frac{1000(16 - 2.33)}{2} = 6888 \text{ lb.}$$

$$q = \frac{3}{2}\frac{V^1}{bd} \qquad bd = \frac{3 \times 6888}{2 \times 120} = 86.0 \text{ sq. in.}$$

The area of a 10 by 14 in. member = 9.5 × 13.5 = 128 sq. in., which furnishes ample shearing resistance.

Deflection, $y = \dfrac{5}{384} \times \dfrac{wl^4}{EI}$ (I for a 10 by 14 in. member = 1947.8)

$$y = \frac{5 \times 16{,}000(16 \times 12)^3}{384 \times 1{,}600{,}000 \times 1947.8} = 0.471 \text{ in.}$$

Allowable deflection = 1/360 of the span.

$$y = \frac{16 \times 12}{360} = 0.534 \text{ in.}$$

Problem. Design a built-up beam using 6 by 8 in. timbers and 4-in. split rings to carry 700 lb. per ft. on a 20-ft. span. Use Douglas fir (1450 f).

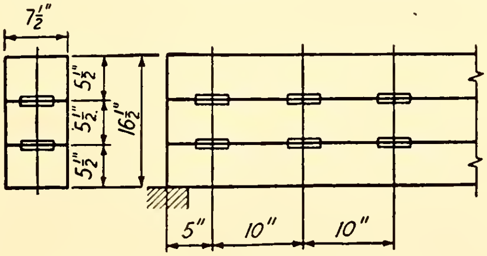


Fig. 78. Built-up beam with split rings.

Assume weight of beam = 35 lb. per ft.

$$f = \frac{M}{s} \quad s = \frac{735 \times 20 \times 20 \times 12}{8 \times 1450} = 304 = \frac{bd^2}{6} \quad d^2 = \frac{304 \times 6}{7.5} \quad d = 15.6 \text{ in.}$$

Use three 6 by 8 in. timbers.

$$V = (700 + 35)10 = 7350 \text{ lb.}$$

$$q = \frac{3 \times 7350}{2 \times 7.5 \times 16.5} = 89 \text{ lb. per sq. in.}$$

Allowable q = 120 lb. per sq. in.

$$\text{Shear at plane of contact} = \frac{VQ}{Ib} = \frac{7350 \times 7.5 \times 5.5 \times 5.5}{\frac{7.5 \times 16.5^3 \times 7.5}{12}} = 78 \text{ lb. per sq. in.}$$

Shear per linear foot = 78 × 12 × 7.5 = 7025 lb.

$$\text{Spacing, 4-in. split rings} = \frac{0.5 \times 5740 \times 2}{7025} \times 12 = 10 \text{ in.}$$

At ¼ point V = 3675 lb., spacing = 20 in., end bearing 7350/390 = 18.9 sq. in., and length = 18.9/7.5 = 2½ in.

Problem. Design the floor system illustrated in Fig. 79.

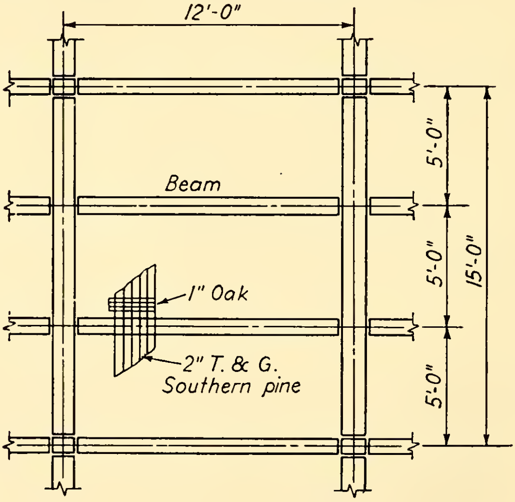


Fig. 79. Floor system.

Loads:

Live load, 80 lb. per sq. ft.
Weight of wood, 40 lb. per cu. ft.

Materials:

Beams and girders, Douglas fir (coast region).
Subflooring, 2-in. T. & G. southern pine.
Flooring, 1-in. T. & G. oak.

Working stresses, Douglas fir beams and girders:

Flexure, 1450 lb. per sq. in.
Compression parallel to grain, 1000 lb. per sq. in.
Compression perpendicular to grain, 390 lb. per sq. in.
Horizontal shear, 120 lb. per sq. in.
Modulus of elasticity, 1,600,000 lb. per sq. in.

Southern pine subflooring:

Flexure, 1000 lb. per sq. in.
Modulus of elasticity, 1,600,000 lb. per sq. in.

Oak flooring:

Flexure, 1000 lb. per sq. in.
Modulus of elasticity, 1,600,000 lb. per sq. in.

Flooring loads:

Live load = 80 lb. per sq. ft.

$$\text{Dead load} = \frac{1.625 \times 40}{12} + \frac{0.75 \times 40}{12} = 7.92 \text{ lb. per sq. ft.}$$

Total load = 80 + 7.92 = 87.92 lb. per sq. ft.

Flooring flexure:

$$\text{Maximum moment} = \frac{87.92 \times 5 \times 5 \times 12}{10} = 2640 \text{ in.-lb.}$$

$$\text{Required section modulus} = \frac{2640}{1000} = 2.64 \text{ in.}^3$$

$$\text{Section modulus provided} = \frac{12 \times 1.625^2}{6} = 5.28 \text{ in.}^3$$

Flooring deflection:

The dead load is increased by 100%, and the total load is $(7.92 \times 2 + 80)5 = 479$ lb. per ft.

$$\text{Maximum deflection} = \frac{3}{640} \times \frac{479 \times 60^3}{1{,}600{,}000 \times \frac{1}{12} \times 12 \times 1.625^3} = 0.071 \text{ in.}$$

$$\text{Allowable deflection} = \frac{60}{360} = 0.167 \text{ in.}$$

Beam loads:

Live load per linear foot = 80 × 5 = 400 lb.
Load due to flooring = 7.92 × 5 = 39.6 lb.

$$\text{Assuming an 8 by 10 in. beam, weight} = \frac{7.5 \times 9.5 \times 40}{12 \times 12} = 19.7 \text{ lb.}$$

Total load = 400 + 39.6 + 19.7 = 459.3 lb. per ft.

Beam flexure:

$$\text{Maximum moment} = \frac{459.3 \times 12 \times 12 \times 12}{8} = 99{,}200 \text{ in.-lb.}$$

$$\text{Required section modulus} = \frac{99{,}200}{1450} = 68.4 \text{ in.}^3$$

$$\text{Section modulus provided} = \frac{7.5 \times 9.5^2}{6} = 112.81 \text{ in.}^3$$

Beam shear:

$$q = \frac{3}{2}\frac{V}{bd} = \frac{3 \times 459.3 \times 6}{2 \times 7.5 \times 9.5} = 58 \text{ lb. per sq. in.}$$

Allowable shear = 120 lb. per sq. in.

Beam deflection:

The dead load is increased by 100%, and the total load is $(59.3 \times 2 + 400)12 = 6230$ lb.

$$\text{Maximum deflection} = \frac{5}{384}\cdot\frac{6230 \times 144^3}{1{,}600{,}000 \times 535.8} = 0.283 \text{ in.}$$

$$\text{Allowable deflection} = \frac{12 \times 12}{360} = 0.40 \text{ in.}$$

Girder loads:

Concentrated loads $= 459.3 \times 12 = 5520$ lb.

$$\text{Assuming a 12 by 14 in. girder, weight} = \frac{11.5 \times 13.5 \times 40}{12 \times 12} = 43.1 \text{ lb. per ft.}$$

Girder flexure:

$$\text{Maximum moment} = 5520 \times 60 + \frac{43.1 \times 15 \times 15 \times 12}{8} = 346{,}600 \text{ in.-lb.}$$

$$\text{Required section modulus} = \frac{346{,}600}{1450} = 239 \text{ in.}^3$$

$$\text{Section modulus provided} = \frac{11.5 \times 13.5^2}{6} = 349.3 \text{ in.}^3$$

Girder shear:

$$V = \frac{43.1 \times 15}{2} + 5520 = 5843 \text{ lb.}$$

$$q = \frac{3 \times 5843}{2 \times 11.5 \times 13.5} = 56.5 \text{ lb. per sq. in.}$$

Allowable shear = 120 lb. per sq. in.

Girder deflection:

Weight of girder increased 100% $= 43.1 \times 2 = 86.2$ lb. per ft. Maximum deflection due to weight of girder is

$$\frac{5 \times 86.2 \times 15 \times 15^3 \times 12^3}{384 \times 1{,}600{,}000 \times 2358} = 0.026 \text{ in.}$$

Weight of beam and flooring increased 100% $= 59.3 \times 2 \times 12 = 1420$ lb.
Live load $= 400 \times 12 = 4800$ lb.
Total concentrated load $= 6220$ lb.
Maximum deflection due to concentrated loads is

$$\frac{23 \times 6220 \times 15^3 \times 12^3}{648 \times 1{,}600{,}000 \times 2358} = 0.342 \text{ in.}$$

Total deflection $= 0.026 + 0.342 = 0.368$ in.

$$\text{Allowable deflection} = \frac{15 \times 12}{360} = 0.50 \text{ in.}$$

127. Beams with Inclined Loads. As the direction of the load acting on rafters and purlins is usually not perpendicular to one of the principal axes, it is necessary to resolve the load into components parallel and perpendicular to the vertical axis of the member.

Example. Design a rafter fastened to purlins and inclined at an angle of 30° as shown in Fig. 80. Span = 12 ft. 0 in.

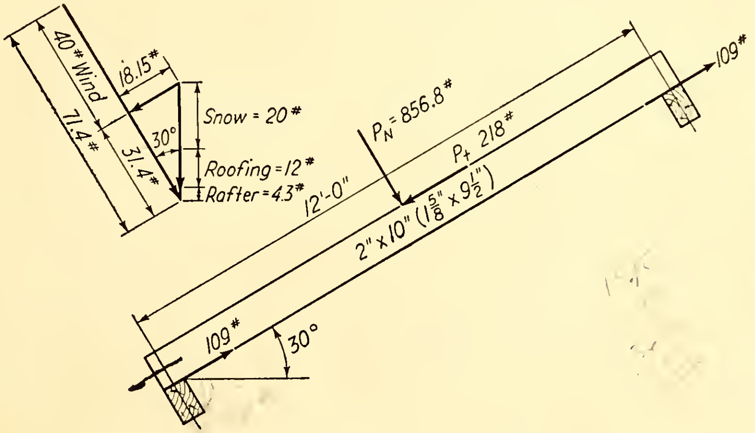

40# Wind
71.4#
18.15#
30°
31.4#
Snow = 20#
Roofing = 12#
Rafter = 4.3#
12'-0"
$P_N = 856.8\#$
P_t 218#
2"×10" ($1\frac{5}{8}$"×$9\frac{1}{2}$")
109#
109#
30°

Fig. 80. Loads acting on inclined rafter.

Vertical loads:

Snow, 20 lb. per ft.
Roofing, 12 lb. per ft.
Rafter, 40 lb. per cu. ft.

Normal loads:

Wind, 40 lb. per ft.

Working stresses:

Flexure, 1100 lb. per sq. in.
Horizontal shear, 110 lb. per sq. in.
Modulus of elasticity, 1,600,000 lb. per sq. in.

Flexure:

Assume a 2 by 10 in. member.

$$\text{Weight of rafters} = \frac{1.625 \times 9.5 \times 40}{12 \times 12} = 4.3 \text{ lb. per ft.}$$

Total vertical load $= 20 + 12 + 4.3 = 36.3$ lb. per ft.
Total normal load $= 36.3 \times 0.866 + 40 = 71.4$ lb. per ft.

$$\text{Moment due to normal load} = \frac{71.4 \times 12 \times 12 \times 12}{8} = 15{,}400 \text{ in.-lb.} \qquad \frac{Wl^2}{8}$$

Moment is also caused by the eccentricity of the tangential force. This force $= 36.3 \times 0.5 = 18.15$ lb. per ft. span

Total tangential force $= 18.15 \times 12 = 218$ lb.

Assuming one-half of this force to be resisted at each purlin, the moment becomes $218/2 \times 9.5 = 1030$ in.-lb. d of rafter

Total moment $= 15{,}400 + 1030 = 16{,}430$ in.-lb.

$$\text{Required section modulus} = \frac{16{,}430}{1100} = 15.0 \text{ in.}^3$$

$$\text{Section modulus provided} = \frac{1.625 \times 9.5^2}{6} = 24.44 \text{ in.}^3 > 15$$

Shear: $w \quad l$

$$V = \frac{71.4 \times 12}{2} = 428 \text{ lb.} \qquad \frac{W}{2}$$

$$q = \frac{3 \times 428}{\underset{b}{2 \times 1.625} \times \underset{h}{9.5}} = 41.7 \text{ lb. per sq. in.} \qquad \frac{3V}{2A}$$

Deflection:

Normal dead load $= 16.3 \times 0.866 = 14.1 \times 2 = 28.2$ lb. per ft.
Normal snow load $= 20 \times 0.866 = 17.3$ lb. per ft.
Normal wind load $= 40$ lb. per ft.
Total load $= (28.2 + 17.3 + 40)12 = 1026$ lb.

$$\text{Maximum deflection} = \frac{5 \times 1026 \times 144^3}{384 \times 1{,}600{,}000 \times 116} = 0.215 \text{ in.}$$

$$\text{Allowable deflection} = \frac{12 \times 12}{360} = 0.4 \text{ in.}$$

Example. Design a purlin to span 16 ft. 0 in. with the roof inclined at 30°. Spacing of purlins = 5 ft. 0 in.

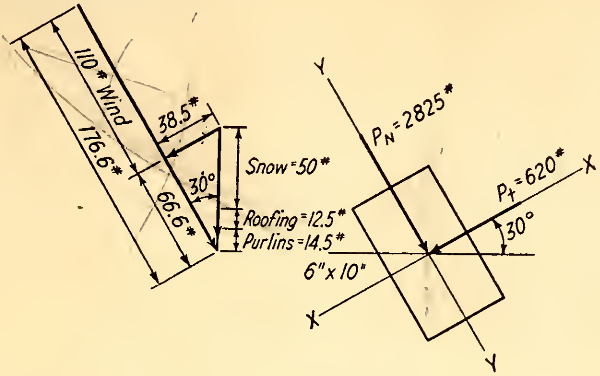


Fig. 81. Inclined purlin.

Vertical loads:
- Snow, 10 lb. per sq. ft.
- Sheathing, 2.5 lb. per sq. ft.
- Weight of purlin = 40 lb. per cu. ft.

Normal loads:
- Wind, 22 lb. per sq. ft.

Working stresses:
- Flexure, 1450 lb. per sq. in.
- Horizontal shear, 120 lb. per sq. in.
- Modulus of elasticity, 1,600,000 lb. per sq. in.

Flexure:

Assume a 6 by 10 in. purlin.

$$\text{Total vertical load} = 12.5 \times 5 + \frac{5.5 \times 9.5}{12 \times 12} \times 40 = 77 \text{ lb. per ft.}$$

$$\text{Total normal load} = 77 \times 0.866 + 22 \times 5 = 176.6 \text{ lb. per ft.}$$

$$\text{Maximum moment} = \frac{176.6 \times 16 \times 16 \times 12}{8} = 68{,}000 \text{ in.-lb.}$$

ACTUAL

$$\text{Section modulus provided} = \frac{5.5 \times 9.5^2}{6} = 82.7 \text{ in.}^3$$

$$\text{Fiber stress due to normal load} = \frac{68{,}000}{\cancel{84.6}\ 82.7} = 803 \text{ lb. per sq. in.}$$

$$\text{Tangential component} = 77 \times 0.5 = 38.5 \text{ lb. per ft.}$$

$$\text{Maximum moment} = \frac{38.5 \times 16 \times 16 \times 12}{8} = 14{,}800 \text{ in.-lb.}$$

$$\text{Section modulus provided} = \frac{9.5 \times 5.5^2}{6} = 47.8 \text{ in.}^3$$

$$\text{Fiber stress due to tangential component} = \frac{14{,}800}{47.8} = 310 \text{ lb. per sq. in.}$$

$$\text{Total fiber stress required} = 803 + 310 = 1113 \text{ lb. per sq. in.}$$

Shear:

$$V = \frac{176.6 \times 16}{2} = 1410 \text{ lb.}$$

$$q = \frac{3 \times 1410}{2 \times 5.5 \times 9.5} = 40.6 \text{ lb. per sq. in.}$$

SAFETY FACTOR?

Deflection:

Normal dead load $= (2.5 \times 5 + 14.5)0.866 \times 2 = 46.7$ lb. per ft.

Normal snow load $= 10 \times 5 \times 0.866 = 43.3$ lb. per ft.

Normal wind load $= 22 \times 5 = 110$ lb. per ft.

Total normal load $= (46.7 + 43.3 + 110)16 = 3200$ lb.

$$\text{Maximum deflection} = \frac{5 \times 3200 \times 16^3 \times 12^3}{384 \times 1{,}600{,}000 \times 402} = 0.458 \text{ in.}$$

$$\text{Allowable deflection} = \frac{16 \times 12}{360} = 0.533 \text{ in.}$$

COLUMNS

128. Classification. Wood columns are divided into three general classes depending upon the ratio of the unsupported length of the column to its least dimension. For short columns, in which the unsupported length divided by the least dimension does not exceed eleven, the full working value in compression may be used, and the formula used in design is $P/A = c$. P/A represents the load divided by the cross-sectional area, and c is the safe unit compressive stress parallel to the grain.

129. Long Columns. The general equation recommended for long columns of uniform cross section is the Euler formula in which

$$\frac{P}{A} = u\frac{\pi^2 E}{\left(\frac{L}{r}\right)^2}$$

where P = maximum load on the column.
A = cross-sectional area.
E = modulus of elasticity.
L = length in inches.
r = radius of gyration.
u = constant depending on end conditions.

Where d is the least dimension of the column, r may be represented by $d/\sqrt{12}$. For pin-ended columns $u = 1$, and for columns with fixed ends $u = 4$.

Substituting 1 for u and $d/\sqrt{12}$ for r in the above equation and introducing a factor of safety of 2.5, gives

$$\frac{P}{A} = \frac{0.329E}{\left(\frac{L}{d}\right)^2}$$

130. Intermediate Columns. For columns of the intermediate class, the Forest Products Laboratory has developed a fourth-power parabolic formula as follows:

$$\frac{P}{A} = c\left[1 - \frac{1}{3}\left(\frac{L}{Kd}\right)^4\right]$$

The curve representing this formula passes through point c on the P/A axis and is tangent to the Euler curve at a point where P/A is two-thirds of c. The value of c is the unit compressive stress for short columns, and the quantity K is a constant depending upon the grade and species of lumber used. K is the value of L/d at which the parabola is tangent to the Euler curve and may be represented by the formula

$$K = \frac{\pi}{2}\sqrt{\frac{E}{5c}} = 0.702\sqrt{\frac{E}{c}}$$

An intermediate column then is one whose L/d ratio is between eleven and K.

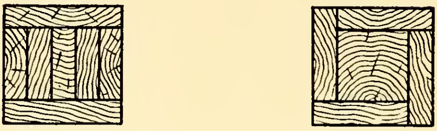
131. Circular Columns. To compute the required size of a round column, design first for a square column and then use a round column having the same cross-sectional area as the square column. In calculating the equivalent area of a long tapered column, the cross-sectional area should be taken at a point one-third of its length from the bottom of the column.

132. Built-Up Columns. Many instances arise in the design of timber structures where it is economical to build up columns of considerable size from smaller pieces by spiking or bolting them together. Table 32 shows the strength of built-up columns in percentage of solid columns of similar grades. These percentages were developed by the Forest Products Laboratory and should be applied to the working stresses for solid columns of equal height, size, and grade. The percentages apply when the width of planks is not more than five times the thickness and when the spikes penetrate three planks. The longitudinal spacing of the spikes should be about six times the thickness of one plank. When the L/d ratio is 10 or greater, the same percentages may be used for columns with butt-joined pieces.

Table 32. Percentage of Solid Column Strength for Built-Up Columns

[Wood Handbook, U. S. Dept. Agr., 1935]

L/d Ratios	*Percentages of Solid-Column Strength*	*L/d Ratios*	*Percentages of Solid-Column Strength*
6	82	18	65
10	77	22	74
14	71	26	82



Edges Tied Together with Cover Plates

Boxed Solid Core

FIG. 82. Built-up columns.

133. Spaced Columns. Spaced columns joined with timber connectors are important in the design of compression members in trusses and other structures. Spaced columns are formed from two or more members separated at their ends and middle point by blocking and joined at the ends by timber connectors.

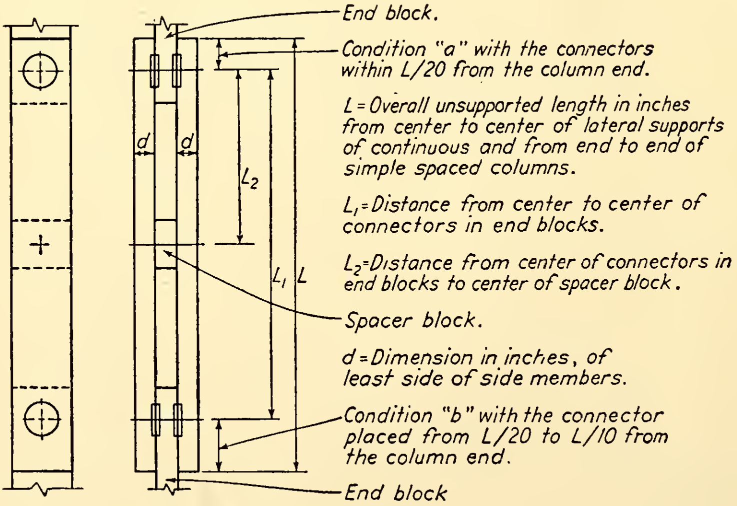

End block.
Condition "a" with the connectors within L/20 from the column end.
L = Overall unsupported length in inches from center to center of lateral supports of continuous and from end to end of simple spaced columns.
L₁ = Distance from center to center of connectors in end blocks.
L₂ = Distance from center of connectors in end blocks to center of spacer block.
Spacer block.
d = Dimension in inches, of least side of side members.
Condition "b" with the connector placed from L/20 to L/10 from the column end.
End block
d
d
L₂
L₁
L

FIG. 83. Spaced column.

In applying the formulas for solid columns to spaced columns it is necessary to introduce a restraint factor depending upon the distance from the center of the connector to the end of the piece.

LONG:

$$\frac{P}{A} = \frac{0.329E \times 2.5}{\left(\frac{l}{d}\right)^2} \text{ for condition } a \text{ (see Fig. 83)}$$

$$\frac{P}{A} = \frac{0.329E \times 3}{\left(\frac{l}{d}\right)^2} \text{ for condition } b$$

INTERMEDIATE:

For condition *a*

$$K_2 = 0.702\sqrt{\frac{2.5E}{c}} = K\sqrt{2.5} = K \times 1.5811$$

$$\frac{P}{A} = c\left[1 - \frac{1}{3}\left(\frac{l}{K_2d}\right)^4\right]$$

For condition *b*

$$K_3 = 0.702\sqrt{\frac{3E}{c}} = K\sqrt{3} = K \times 1.7320$$

$$\frac{P}{A} = c\left[1 - \frac{1}{3}\left(\frac{l}{K_3d}\right)^4\right]$$

The above formulas are based on a spacer block within the middle 10 per cent of the length of the column and of end blocks so placed that they provide the proper end and edge distances for the connectors. Spacer and end blocks should be as thick as the individual column member. For compression members of trusses, the panel point may be considered as the end of the column and the web members as end blocks, provided that the panel point is supported laterally.

The connectors at each end of a spaced column between the contacting face of the end block and an individual member of the column should be of a size and number to provide a load capacity in pounds equal to the product of the cross-sectional area of one of the members and the appropriate end-spacer block constant in Table 33.

Table 33. Constants for Use in Determining the Load Capacity of Connectors in Spaced Columns

[Wood Structural Design Data, Supplement 4, National Lumber Manufacturers' Association, 1939]

L/d Ratio of Individual Member in the Spaced Column	*End-Spacer Block Constant*		
	Group A Connector Loads	Group B Connector Loads	Group C Connector Loads
0 to 11	0	0	0
15	35	31	29
20	79	69	66
25	122	107	103
30	166	145	140
35	210	184	177
40	253	222	214
45	297	260	250
50	340	298	287
55	384	337	324
60 to 80	428	375	361

ILLUSTRATIVE PROBLEM

Problem. Design a column 12 ft. 9 in. high to carry a load of 120,000 lb. Use Douglas fir with a working stress of 1450 lb. per sq. in.

$$\text{Constant } K = 0.702\sqrt{\frac{E}{c}} = 0.702\sqrt{\frac{1{,}600{,}000}{1450}} = 23.3$$

Assume a 10 by 10 in. column.

$$\frac{L}{d} = \frac{12.75 \times 12}{9.5} = 16.1$$

Since L/d is greater than eleven but less than K, the column is in the intermediate class.

$$\frac{P}{A} = c\left[1 - \frac{1}{3}\left(\frac{L}{Kd}\right)^4\right] = 1450\left[1 - \frac{1}{3}\left(\frac{16.1}{23.3}\right)^4\right] = 1340 \text{ lb. per sq. in.}$$

Allowable load = 1340 × 9.5 × 9.5 = 120,080 lb.

Assume that the column supports a 10 by 14 in. girder and a cast iron pintle which acts as the base for a 10 by 10 in. column 20 ft. long. Using a working stress in compression of 12,000 lb. per sq. in. for the pintle, find its dimensions.*

---

* "Design of Modern Steel Structures," by L. E. Grinter, The Macmillan Co., 1941.

Load on pintle:

$$L/d \text{ of column} = \frac{20 \times 12}{9.5} = 25.3.$$

Since L/d is greater than K the formula for long columns applies.

$$\frac{P}{A} = \frac{0.329E}{(L/d)^2} = \frac{0.329 \times 1,600,000}{25.3^2} = 822 \text{ lb. per sq. in.}$$

$$\text{Allowable load} = 822 \times 9.5 \times 9.5 = 74,200 \text{ lb.}$$

$$\text{Required area of pintle} = \frac{74,200}{12,000} = 6.18 \text{ sq. in.}$$

Try a pintle with a stem having an outside diameter of 3 in. and an inside diameter of 1 in.

$$\text{Area} = \frac{\pi}{4}(3^2 - 1^2) = 6.28 \text{ sq. in.}$$

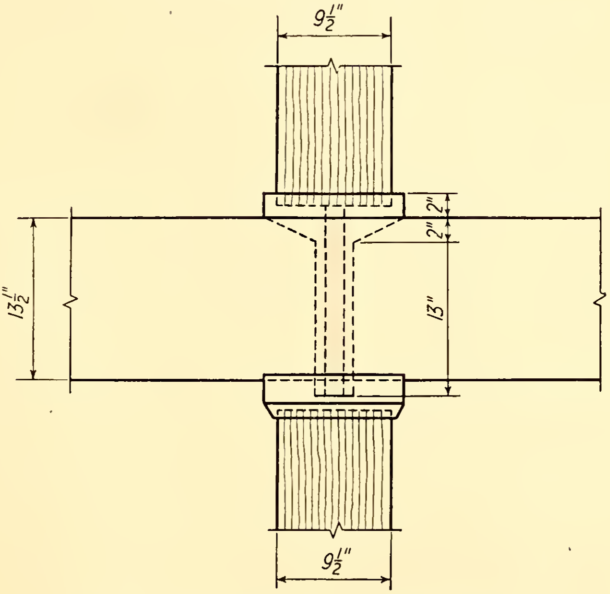


Fig. 84. Column supported on cast-iron pintle.

134. Members with Combined Bending and Axial Loads. The design of members subjected to both axial tension and bending is limited by the formula $f = P/A + Mc/I$. However, special formulas are necessary for the design of compression members subjected to axial and bending loads because the allowable compressive stress differs from the allowable stress in tension.

The following formulas for columns with end loads, side loads, and eccentricity were derived by J. A. Newlin, former specialist in the mechanics of timber, Forest Products Laboratory, and have been published in the Building Standards Monthly, December, 1940. They have also been published by the Timber Engineering Company in a document by J. E. Myer.

SYMBOLS USED

f = allowable extreme fiber stress in bending in pounds per square inch.

L = length of span or column in inches.

d_b = dimension of a rectangular column in the direction of the side load.

e = eccentricity in inches.

$S_b = M/S$ = unit bending stress due to side load.

k = the ratio of the unit bending stress due to side load to the unit compressive stress due to end load when side load is proportional to end load.

c_b = allowable stress in compression parallel to the grain, in pounds per square inch, which would be permitted for the column if axial compression stress only existed; that is, the safe working stress for the appropriate L/d_b ratio.

Q = allowable stress in compression parallel to grain, in pounds per square inch, which is permitted for the column when side eccentric load or both are combined with end load.

COLUMNS WITH L/d_b = 11 OR LESS

Combined end and side load:

$$Q = \frac{c_b(f - S_b)}{f}$$

Eccentrically loaded columns:

$$Q = \frac{c_b f}{f + c_b\left(\frac{6e}{d_b}\right)}$$

Side load proportional to end load:

$$Q = \frac{c_b f}{f + c_b k}$$

Combined end load, side load, and eccentricity:

$$Q = \frac{c_b(f - S_b)}{f + c_b\left(\dfrac{6e}{d_b} + k\right)}$$

COLUMNS WITH L/d_b = 20 OR MORE

Combined end and side load:

$$Q = \frac{f + c_b}{2} - \sqrt{\left(\frac{f + c_b}{2}\right)^2 - c_b(f - S_b)}$$

Eccentrically loaded:

$$Q = \frac{f + c_b\left(1 + \dfrac{15e}{2d_b}\right)}{2} - \sqrt{\left(\frac{f + c_b\left(1 + \dfrac{15e}{2d_b}\right)}{2}\right)^2 - c_bf}$$

Side load proportional to end load:

$$Q = \frac{f + c_b(1 + k)}{2} - \sqrt{\left(\frac{f + c_b(1 + k)}{2}\right)^2 - c_bf}$$

Combined end load, side load, and eccentricity:

$$Q = \frac{f + c_b\left(1 + \dfrac{15e}{2d_b} + k\right)}{2} - \sqrt{\left(\frac{f + c_b\left(1 + \dfrac{15e}{2d_b} + k\right)}{2}\right)^2 - c_b(f - S_b)}$$

COLUMNS WITH L/d_b BETWEEN 11 AND 20

When L/d_b is between 11 and 20, the stress should be assumed to vary as a straight line. The value of Q computed by the formula for $L/d_b = 20$ will be found to be only slightly less than the value found by interpolation between the two formulas, and for all practical purposes it will usually be unnecessary to calculate Q for $L/d_b = 11$ and then interpolate.

If the top chord of a roof truss supports roof rafters between the panel points, it is subjected to combined axial compression and side loading. If the top chord is continuous through the panel point, the bending moment may be computed from the formula $WL/10$. If a splice occurs within a span, the same bending moment $WL/10$ may be used for this span also.

Example. As a specific example of the design of a top chord member of a truss assume the following:

Flat top truss, panel length 8 ft.
Truss spacing 16 ft. on centers.
Uniformly loaded top chord.
Total dead and live loads, 40 lb. per sq. ft.
Load in top chord, 20,000 lb. compression.

Working stresses:

Flexure, 1450 lb. per sq. in.
Compression, 1200 lb. per sq. in.
Modulus of elasticity, 1,600,000 lb. per sq. in.

Assume a member 5½ by 11½ in.

$$\frac{L}{d_b} = \frac{96}{11.5} = 8.35$$

Because this ratio is less than 11 the following formula applies:

$$Q = \frac{c_b f}{f + c_b k}$$

$$k = \frac{\frac{M}{S}}{\frac{P}{A}} = \frac{\frac{40 \times 16 \times 8 \times 96}{10 \times 124}}{\frac{20,000}{5.5 \times 11.5}} = \frac{396}{316} = 1.25$$

$$Q = \frac{1200 \times 1450}{1450 + 1200 \times 1.25} = 590 \text{ lb. per sq. in.}$$

Since the actual load of 316 lb. per sq. in. is less than the computed permissible load of 590 lb. per sq. in., the column is adequate.

Example. To illustrate the method for determining the adequacy of a column whose L/d_b ratio is between 11 and 20, assume the same conditions as those given in the previous example, but consider the panel length as 12 ft.

$$\frac{L}{d_b} = \frac{144}{11.5} = 12.5$$

One of the following formulas applies. If $L/d_b = 20$ or more,

$$Q = \frac{f + c_b(1 + k)}{2} - \sqrt{\left(\frac{f + c_b(1 + k)}{2}\right)^2 - c_b f}$$

If $L/d_b = 11$ or less,

$$Q = \frac{c_b f}{f + c_b k}$$

If the formula for an L/d_b ratio equal to 20 or more is used and the load-carrying capacity of the member is found sufficient, no further computations are necessary.

$$k = \frac{\frac{M}{S}}{\frac{P}{A}} = \frac{\frac{40 \times 16 \times 12 \times 144}{10 \times 124}}{\frac{20{,}000}{5.5 \times 11.5}} = \frac{893}{316} = 2.83$$

$c_b = 1180$ lb. per sq. in. (from column formula)

$$Q = \frac{1450 + 1180(1 + 2.83)}{2} - \sqrt{\left(\frac{1450 + 1180(1 + 2.83)}{2}\right)^2 - 1180 \times 1450}$$

$$= 2985 - 2680 = 305 \text{ lb. per sq. in.}$$

The permissible stress of 305 lb. per sq. in. is very little less than the stress developed, so the member may be considered adequate.

PROBLEMS

1. Design a timber beam simply supported at its ends to carry a uniform load of 400 lb. per ft. and concentrated loads of 3000 lb. and 2000 lb. at the quarter points when the span is 16 ft. Use 1700 f Douglas fir.

2. If 3 by 10 in. joists span 17 ft. and are placed 24 in. on centers, what is the maximum safe load in pounds per square foot that can be placed on the floor? Use No. 1 dense structural southern pine for the joists and 1-in. hemlock subflooring with a finish floor of 1-in. oak.

3. Design a beam to span 10 ft. and carry 25,000 lb. concentrated at the center. Check shear and deflection, and design the bearing plates at the reaction. Use 1450 f redwood.

4. An 8 by 10 in. timber beam with a span of 12 ft. supports a concentrated load P at the mid-span. The beam is No. 1 structural longleaf pine. Allowing for the weight of the beam, what is the maximum safe value of P?

5. Using bolts and connectors, and 10 by 10 in. timbers, design a built-up beam to carry 1000 lb. per ft. over a span of 20 ft. Include the weight of the beam and use 1300 f Douglas fir.

6. A 6 by 6 in. column is 10 ft. long. What safe load will it carry, if the column is tidewater red cypress with a working value in compression of 1450 lb. per sq. in.?

7. Design a column 16 ft. long to carry a load of 100,000 lb. Use 1450 c Douglas fir.

8. A round column 12 ft. high supports an axial load of 12,000 lb. Using 1100 c redwood, what size column is required?

9. Determine the load-carrying capacities of the following columns: (*a*) 12 by 12 in. by 11 ft. 0 in. 1325 c Douglas fir, (*b*) 8 by 10 in. by 16 ft. 0 in. 1200 c southern cypress, (*c*) 6 by 10 in. by 14 ft. 0 in. 1200 c oak, and (*d*) 10 by 12 in. by 18 ft. 0 in. 1400 c dense shortleaf southern pine.

10. Design a 16 ft. 0 in. column of 1400 c dense shortleaf southern pine to carry an axial load of 12,120 lb. and a side load of 20 lb. per sq. ft. The columns are spaced 16 ft. 0 in. on centers.

CHAPTER VII

WOOD TRUSSES

135. Types of Trusses. Figure 85 illustrates the types of wood trusses most commonly used in building construction. These are usually built of wood throughout with the exception of the metal used as bolts and connectors in framing the joints and splices. The top and bottom chords and the diagonal tension members are made up of two or more spaced members with the web compression member as a solid piece. The bowstring trusses often have a two-member top chord consisting of relatively thin pieces of lumber glued together to form the required thickness.

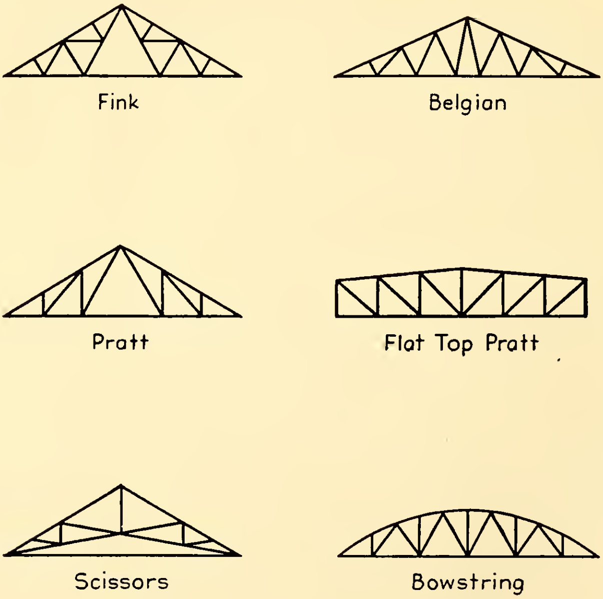


Fig. 85. Types of wood trusses.

136. Design Procedure. In the design of a wood truss there are certain chronological steps that when followed will reduce the amount of work involved. The general procedure is outlined below:

1. Determine the live loads acting on the truss.

2. Compute the dead loads, including roofing, sheathing, rafters, purlins, weight of truss, and any other stationary loads.
3. Determine the stresses in the members.
4. Select the species and grade of lumber to be used. This selection depends upon the locality in which the structure is built and the relative cost of the various species that are usable for structural purposes.
5. Compute the size of each member required to carry the stress due to the combination of loads causing the greatest stress.
6. Select the size and type of connector. When possible, it is best to keep the same size and type throughout the truss.
7. Design the joint that carries the greatest load. The placement of the rings in this joint will usually determine the size of the members and will facilitate the design of the remaining joints. Determine the member in the joint that has the loads acting on it at an angle to the grain and design for the largest load first.
8. Calculate the permissible reductions in the spacing of connectors and the end distances of the members.
9. Check the edge distance of the connectors and the size of the members.

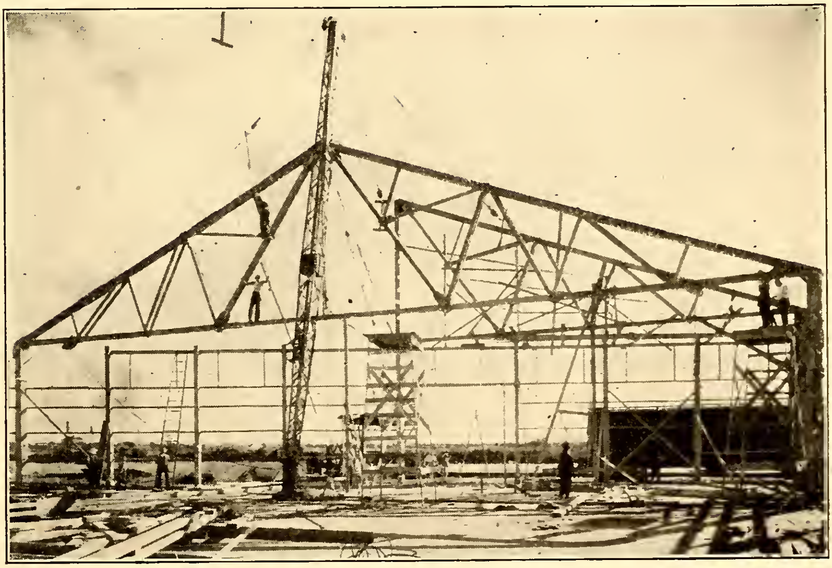


FIG. 86. Erection of 100-ft. Fink trusses.

137. Dead and Live Loads. The total load for which a truss must be designed consists of the weight of the roofing material, rafters, and purlins and the weight of the truss, the snow load, and the pressure of the wind. The following table may be used in estimating the weights in pounds per square foot of roof surface of various roofing material.

Table 34. Weights of Roofing Material

Roofing Material	*Approximate Weight (Pounds per Square Foot)*
Corrugated asbestos, $\frac{1}{4}$ in. thick	3
Corrugated asbestos, $\frac{3}{8}$ in. thick	4.5
Corrugated steel, No. 22 gage (U. S. Standard)	1.5
Corrugated steel, No. 20 gage (U. S. Standard)	1.8
Corrugated steel, No. 18 gage (U. S. Standard)	2.3
Felt and asphalt	2
Slag roof, 4-ply with cement and sand	4
Slate, $\frac{1}{8}$ in. thick, 3-in. double lap	4.5
Slate, $\frac{3}{16}$ in. thick, 3-in. double lap	6.75
Tiles	10–20
Wood, shingles	2

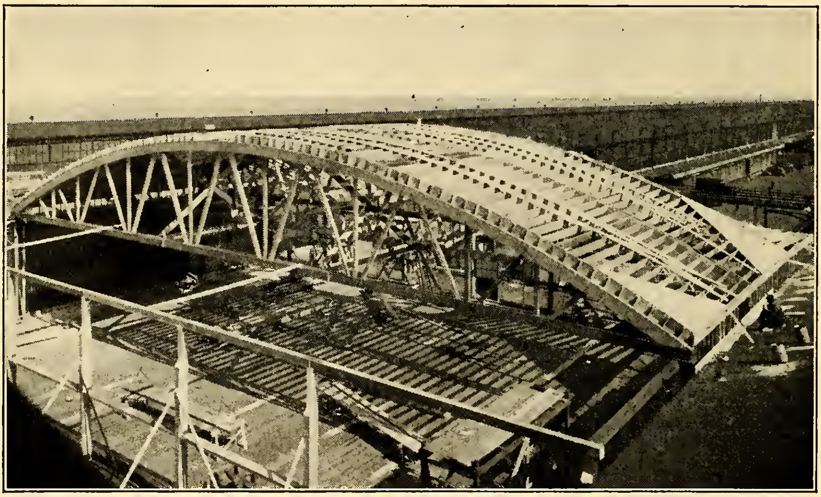


Fig. 87. 111-ft. bowstring trusses designed and built by McKeown Bros., Chicago, Illinois; 2½-in. split rings used between web members and chords and 4-in. flange shear plates at heel joints.

The weights of wood in pounds per 1000 board feet given in Table 35 are based on the nominal size, and in estimating the weight of any given quantity of lumber it is necessary to multiply the weights in the table by the ratio of the actual size of each piece to the nominal size. For instance, to find the actual weight of 1000 board feet of larch 2 by 4 in. boards, multiply 3000 by the ratio $(1.625 \times 3.625)/(2 \times 4)$. The result is 2210 lb.

Table 35. Average Weights of Various Species of Wood

Species	Weight (Pounds per Cubic Foot) Green	Weight (Pounds per Cubic Foot) Air-Dry (12 Per Cent Moisture Content)	*Weight (Pounds per 1000 Board Feet)* Nominal Size (12 Per Cent Moisture Content)
Ash, white	48	41	3420
Beech	54	45	3750
Birch	57	44	3670
Cedar, western red	27	23	1920
Chestnut	55	30	2500
Cypress, southern	51	32	2670
Douglas fir (coast region)	38	34	2830
Douglas fir (inland region)	36	31	2580
Elm, rock	53	44	3670
Gum, black	45	35	2920
Gum, red	50	34	2830
Gum, tupelo	56	35	2920
Hemlock, eastern	50	28	2330
Hemlock, western	41	29	2420
Hickory, pecan	62	45	3750
Hickory, true	63	51	4250
Larch, western	48	36	3000
Maple, hard	56	44	3670
Oak, red	64	44	3670
Oak, white	63	47	3920
Pine, Norway	42	34	2830
Pine, longleaf	55	41	3420
Pine, shortleaf	52	36	3000
Poplar, yellow	38	28	2330
Redwood	50	28	2330
Spruce, eastern	34	28	2330
Spruce, Sitka	33	28	2330
Walnut, black	58	38	3170

The weights of wood trusses in pounds per square foot of horizontal surface may be found by using the following formulas developed by the writer:

$$\text{Pitched roof trusses} = 0.064S$$

$$\text{Flat top Pratt trusses} = 0.043S + 1.75$$

$$\text{Bowstring trusses} = 0.038S + 0.6$$

where S = the span of the truss in feet.

The snow load, which depends upon the location and the roof pitch of the structure, ranges from practically 0 to 50 lb. per sq. ft. of roof sur-

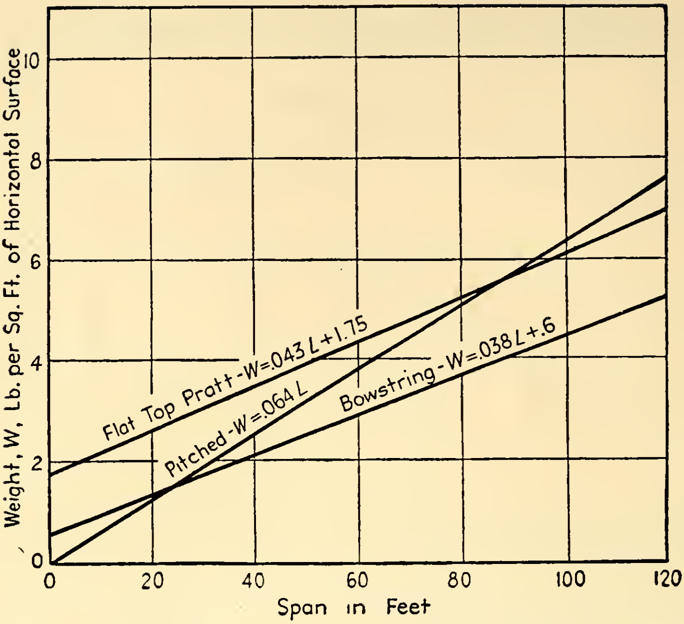


CHART 34. Curves for estimating weights of wooden roof trusses.

face. For roofs having a ⅓ to ¼ pitch, a snow load of 15 to 20 lb. per sq. ft. is commonly used in the Central States and 5 to 10 lb. per sq. ft. in the Southern States. For flat roofs these values should be doubled, and for steep roofs they may be decreased by 50 per cent.

The wind loads vary throughout the country. Most building codes specify the amount to use in designing a structure. The most common value is 20 lb. per sq. ft. of vertical surface. The component of the wind load normal to the roof may be found by Duchemin's formula.

$$P_n = P_h \left(\frac{2 \sin \theta}{1 + \sin^2 \theta} \right)$$

where P_h = horizontal pressure per square foot on a vertical surface.
P_n = normal pressure per square foot on a sloping surface.
θ = angle of inclination of the roof surface.

The above formula fails to take into account several factors including (1) the suction on the leeward roof, (2) the suction on the windward side of the roof when the angle of inclination is small, and (3) internal pressure or suction. A great number of model studies have shown that suc-

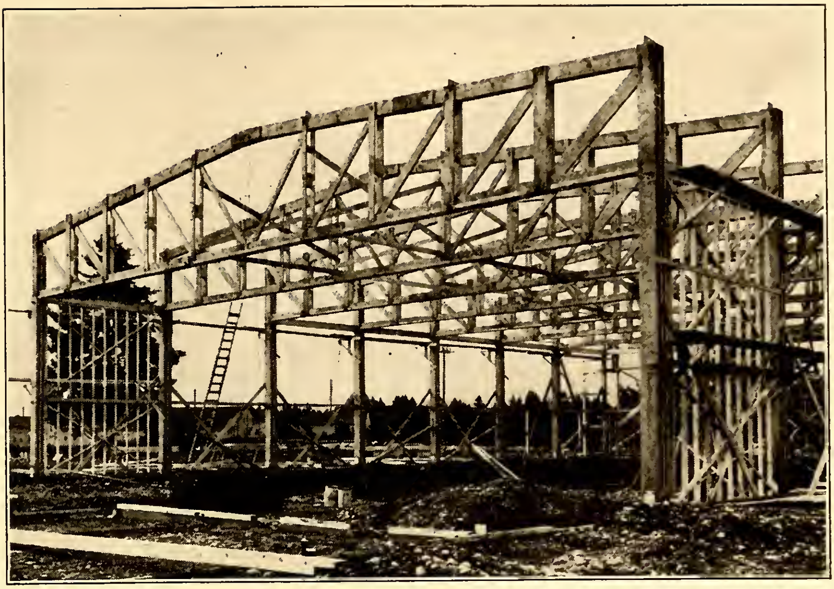


FIG. 88. 75-ft. Pratt trusses with spaced columns made an integral part of the truss.

tion does exist on the leeward side regardless of the slope of the roof and that for slopes less than 30° the wind acts as suction on the windward slope. Furthermore, experimental results have shown that the following relationship exists:

$$p = cq$$

where p = wind pressure in pounds per square foot.
c = coefficient depending upon size, shape, and position of the structure in the wind.
q = velocity pressure = $0.00256V^2$, where V is the velocity of the wind in miles per hour.

Values of q for various wind speeds may be found in Table 36.

In the final report * of subcommittee 31 of the Committee on Steel of the Structural Division of the American Society of Civil Engineers, recommendations are made for determining the wind pressures on plane surfaces inclined to the wind. The values suggested are based on activities of the committee covering a period of ten years in which all important experimental results of wind pressures on buildings were studied.

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* American Society of Civil Engineers, N. Y., *Transactions*, Vol. 66, No. 8, Part 2, pp. 1713–1739, 1940.

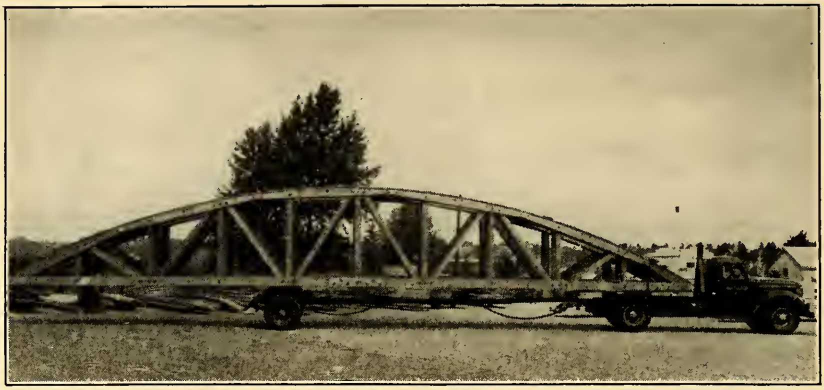


FIG. 89. Modified arch-type bowstring truss being transported by specially built truck.

Table 36. Velocity Pressures q for Various Wind Speeds

Wind Speed (Miles per Hour)	q *(Pounds per Square Foot)*	*Wind Speed (Miles per Hour)*	q *(Pounds per Square Foot)*
5	0.064	65	10.81
10	0.256	70	12.53
15	0.575	75	14.39
20	1.023	80	16.37
25	1.600	85	18.48
30	2.302	90	20.72
35	3.133	95	23.08
40	4.092	100	25.58
45	5.179	105	28.20
50	6.394	110	30.95
55	7.737	115	33.83
60	9.208	120	36.83

The recommendations of the committee for values to use in design may be divided into three parts, namely, vertical plane surfaces, sloping surfaces, and curved surfaces.

I. Vertical plane surfaces of 300 ft. or less.
   A. Wind velocity of 77.8 miles per hour or a velocity pressure q of 15.5 lb. per sq. ft.
   B. External pressure on windward side, $p = 0.8q = 12.4$ lb. per sq. ft.

II. Sloping surfaces.
   $p = cq$, where q is 15.5 lb. per sq. ft. and c depends on θ, the angle of inclination. Values of c may be taken from Chart 35. Two values are shown. One represents values for an airtight building, which is normally

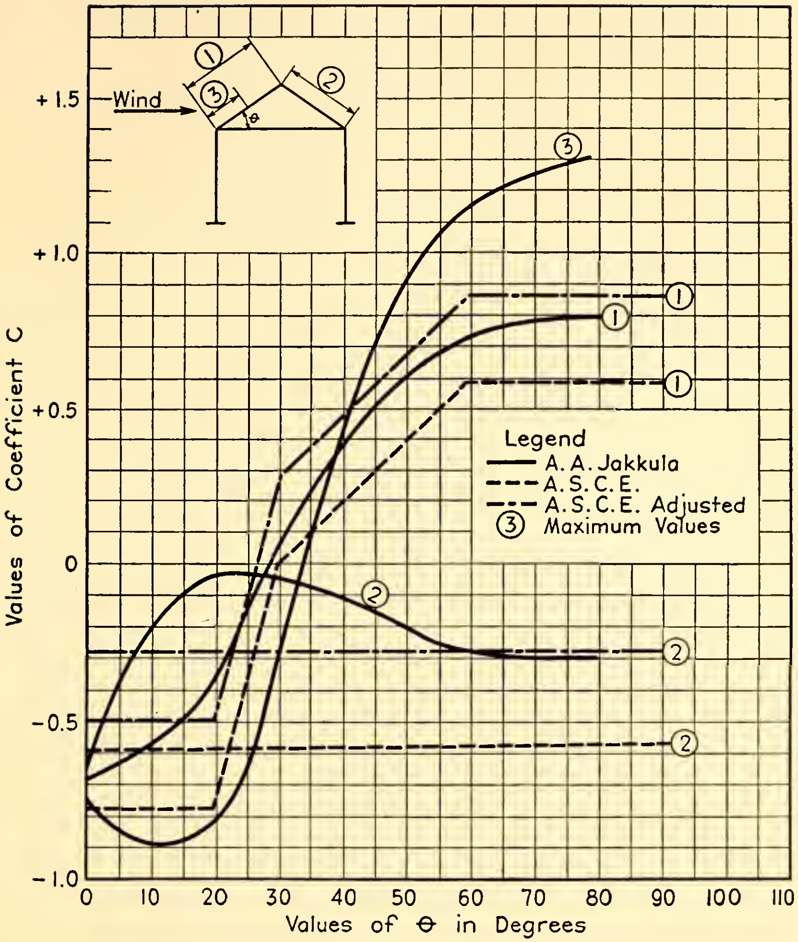


CHART 35. Values of c in the formula $p = cq$.

an impossible condition, and the other represents adjusted values of c based on external pressure or suction and an internal suction of 4.5 lb. per sq. ft.

III. Curved surfaces not more than 300 ft. above ground level. Values are usually based on r, the ratio of R/s, where R is the rise and s is the span.
  - *A*. Windward quarter of roof arc:
    - *a*. For roofs resting on elevated vertical supports.
      1. When r is less than 0.20, suction = 12 lb. per sq. ft.
      2. When r is greater than 0.20, pressure $p = 30r - 6$.
      3. Alternatively, when r is between 0.20 and 0.35, $p = 8$ or -28. A negative sign indicates suction.
    - *b*. For roofs starting from ground level. For all values of r, pressure $p = 19r$.
  - *B*. Central half of roof arc.
    - *a*. Roofs resting on elevated vertical supports. Suction of $p = 15r + 11$.
    - *b*. Roofs starting from ground level. Suction of 11 lb. per sq. ft.
  - *C*. For the leeward quarter of roof arc, suction is 9 lb. per sq. ft.

For buildings with 30 per cent or more wall spaces open, or which could be opened, designs may be based on an internal pressure of 12 lb. per sq. ft. or internal suction of 9 lb. per sq. ft.

For buildings with percentages n of openings from 0 to 30 per cent of wall space, use an internal pressure $p = 4.5 + 0.25n$ or an internal suction $p = 4.5 + 0.15n$.

For buildings nominally airtight, use a pressure or suction of 4.5 lb. per sq. ft.

138. Design of a 50-Ft. Fink Truss. A 50-ft. wooden Fink truss is to be designed to carry 6 by 10 in. Douglas fir purlins at the panel points which support 2-in. T. & G. southern pine sheathing and $\frac{3}{16}$-in. slate roofing. The trusses are spaced 16 ft. on centers and have the dimensions which are shown in Fig. 90.

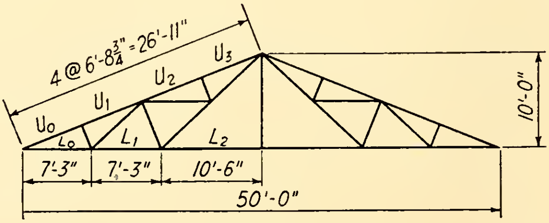


FIG. 90. 50-ft. Fink truss.

+ from p. 167

DEAD LOADS

Weight of truss per square foot of horizontal surface $= 0.064 \times 50 = 3.2$ lb.

Total weight of truss $= 3.2 \times 50 \times 16 = 2560$ lb.

Load per panel point $= \dfrac{2560}{8} = 320$ lb.

Weight of purlins $= \dfrac{(5.5 \times 9.5)^2 \times 16 \times 2830}{6 \times 10 \times 12 \times 1000} = 172$ lb. (use 200 lb.)

Load per panel point $= 200$ lb.

Weight of sheathing $= 1.625 \times 3 = 4.87$ lb. per sq. ft.

Load per panel point $= 4.87 \times 16 \times 6.73 = 524$ lb. (use 530 lb.)

Weight of roofing $= 6.75$ lb. per sq. ft.

Load per panel point $= 6.75 \times 16 \times 6.73 = 727$ lb.

LIVE LOADS

Snow load $= 20$ lb. per sq. ft.

Load per panel point $= 20 \times 16 \times 6.73 = 2150$ lb.

Wind load: From Chart 35,

Windward slope $P_n = -0.36 \times 15.5 = -5.58$ lb. per sq. ft.

Leeward slope $P_n = -0.29 \times 15.5 = -4.50$ lb. per sq. ft.

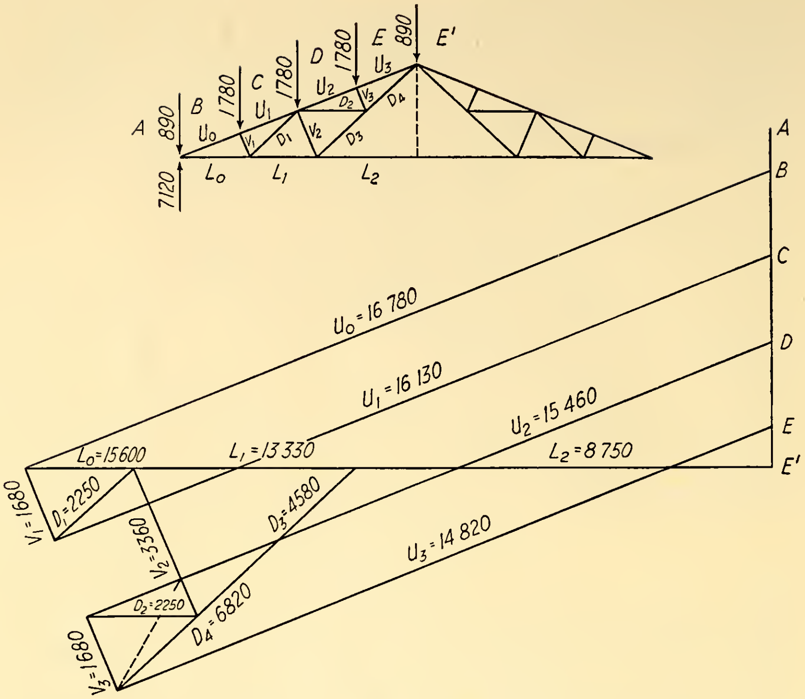
Load per panel point:

Windward slope $= -5.58 \times 16 \times 6.73 = -600$ lb.

Leeward slope $= -4.50 \times 16 \times 6.73 = -485$ lb.

	Total Panel Loads
Slate	730
Sheathing	530
Purlins	200
Truss	320
Total dead panel load	1780
Snow	2150
Wind, windward slope	−600
leeward slope	−485

Figure 91 is the dead load stress diagram. The stresses due to snow load may be calculated by multiplying the dead load stresses by the ratio 2150/1780.


890
1780
1780
1780
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A
B
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D
E
E'
U_0
U_1
U_2
U_3
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D_1
V_2
D_2
V_3
D_3
D_4
L_0
L_1
L_2
7120
A
B
C
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E
E'
U_0 = 16 780
U_1 = 16 130
U_2 = 15 460
L_0 = 15600
L_1 = 13 330
L_2 = 8 750
V_1 = 1680
D_1 = 2250
V_2 = 3360
D_3 = 4580
U_3 = 14 820
D_2 = 2250
D_4 = 6820
V_3 = 1680

FIG. 91. Dead load stress diagram.

STRESSES IN MEMBERS

The stresses in the members are listed in Table 37.

Table 37. Stresses in Members

Member	*Dead Load*	*Snow Load*	*Dead Load + Snow Load*	*Wind Load*
U_0	−16,780	−20,250	−37,030	+4,910
U_1	−16,130	−19,500	−35,630	+4,910
U_2	−15,460	−18,690	−34,150	+4,910
U_3	−14,820	−18,020	−32,840	+4,910
L_0	+15,600	+18,850	+34,450	−4,540
L_1	+13,330	+16,110	+29,440	−3,730
L_2	+8,750	+10,570	+19,320	−2,120
V_1	−1,680	−2,030	−3,710	+600
V_2	−3,360	−4,060	−7,420	+1,200
V_3	−1,680	−2,030	−3,710	+600
D_1	+2,250	+2,720	+4,970	−810
D_2	+2,250	+2,720	+4,970	−810
D_3	+4,580	+5,550	+10,130	−1,610
D_4	+6,820	+8,240	+15,060	−2,430

Because of the slope of the roof the wind load becomes suction on both sides and actually causes a reversal of stress in the members of the truss. In this particular example the reversal is small and may be neglected. Normally, however, a member should be designed for both conditions of stress because, if the compressive stress due to wind is high in comparison with the tensile stress due to dead or snow load, it may govern the design.

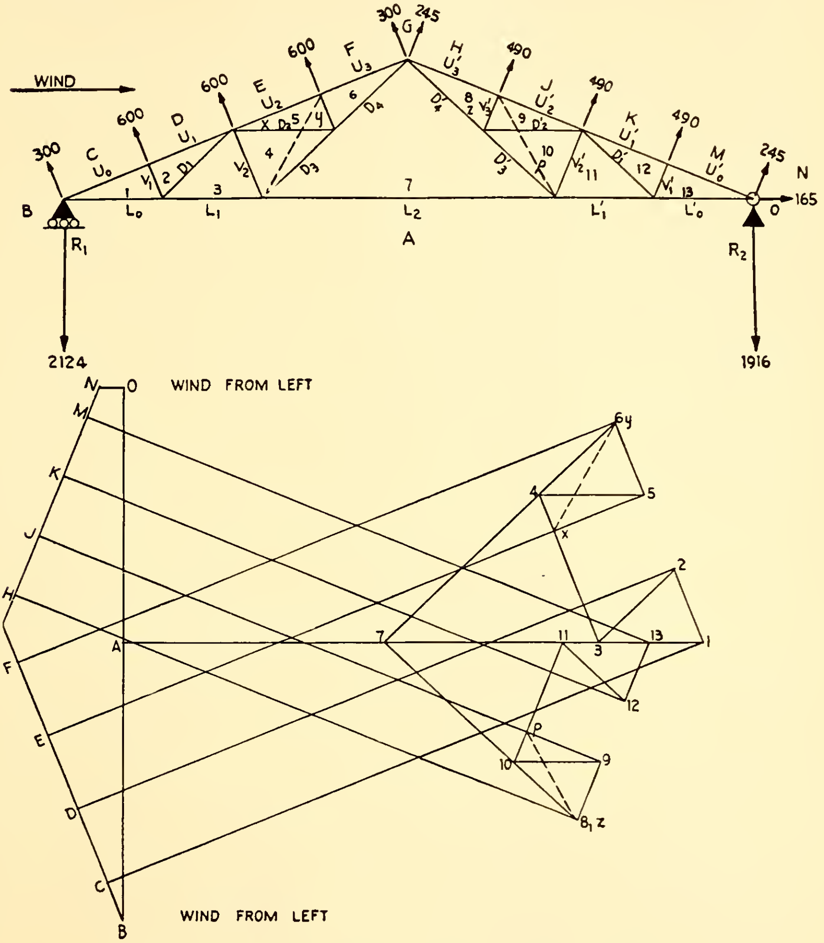

WIND
300
600
600
600
300
245
490
490
490
245
165
B
C
D
E
F
G
H
J
K
M
N
O
A
X
P
U₀
U₁
U₂
U₃
U'₃
U'₂
U'₁
U'₀
L₀
L₁
L₂
L'₁
L'₀
V₁
V₂
V'₃
V'₂
V'₁
D₁
D₂
D₃
D₄
D'₄
D'₃
D'₂
D'₁
R₁
R₂
2124
1916
1
2
3
4
5
6
7
8
9
10
11
12
13
WIND FROM LEFT
6,y
8,z
WIND FROM LEFT

FIG. 92. Wind load stress diagram (wind from left).

In this example, then, only the stresses due to dead and snow loads will be considered, and the allowable working stresses will be increased 15 per cent when designing for the combination of dead and snow loads.

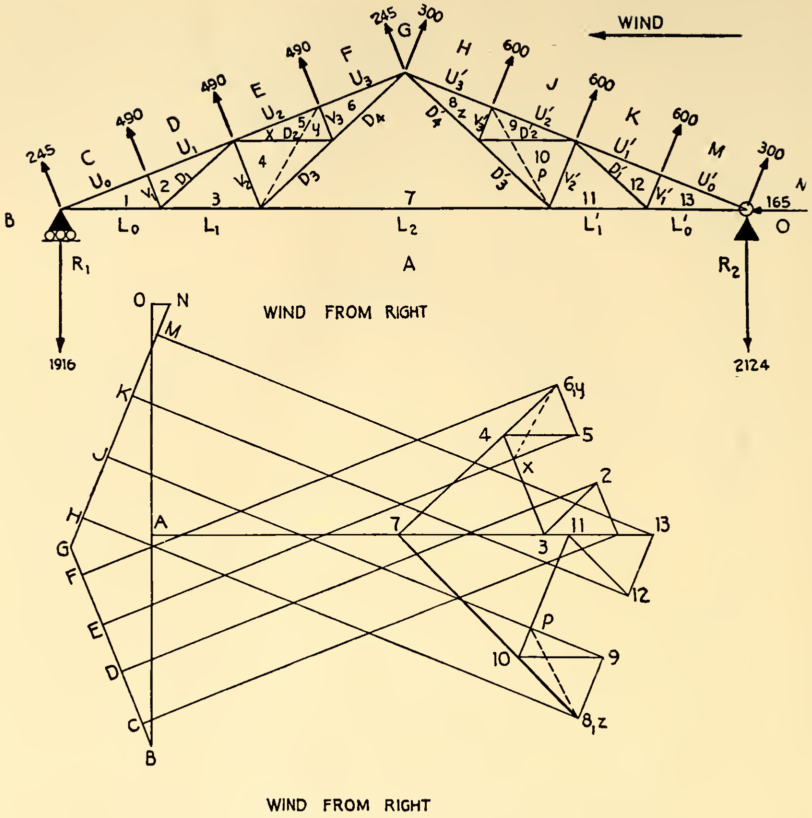

WIND
245
300
G
490
F
U3
H
600
J
E
U2
6
U'3
D'4
8 z
V'3
9
D'2
U'2
K
M
D
U1
X
D2
5/y
V3
D4
10
P
V'2
U'1
D'1
12
V'1
13
U'0
C
U0
1
V1
2
D1
4
V2
D3
D'3
11
165
N
B
3
7
L0
L1
L2
L'1
L'0
O
R1
A
R2
1916
2124
O
N
M
WIND FROM RIGHT
K
J
H
G
F
E
D
C
B
A
6,y
4
5
X
2
7
3
11
13
12
P
10
9
8,z
WIND FROM RIGHT

Fig. 93. Wind load stress diagram (wind from right).

Select Douglas fir, coast region with working stresses as follows:

	Dead Load	*Dead Load* + *Snow Load*
Extreme fiber in bending	1,450	1,670
Horizontal shear	120	138
Compression perpendicular to grain	390	450
Compression parallel to grain	1,200	1,380
Modulus of elasticity	1,600,000	1,600,000

DESIGN OF JOINTS

The lower chord joint between L_1 and L_2 will be designed first. The lower chord will be assumed to be two members 3 in. thick placed between a single compression member V_2 and a double diagonal tension member D_3. V_2 and D_3 transmit the maximum loads to the chord, and the design of this joint will determine the width of L_1 and L_2.

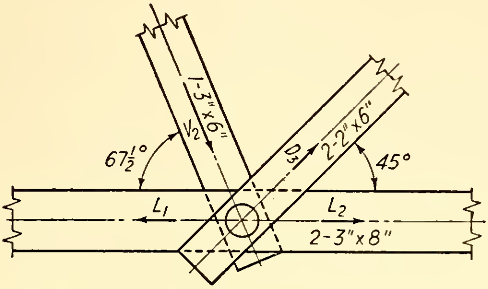


FIG. 94. Lower-chord joint for 50-ft. Fink truss.

Connector Data and Loads	*D_3 to Chord*	*V_2 to Chord*
Size and type	4-in. split ring	4-in. split ring
Number of faces	2	2
Thickness of piece	$2\frac{5}{8}$ in.	$2\frac{5}{8}$ in.
Species group	*B*	*B*
Moisture content condition	Dry	Dry
Basic design load	5,635	5,635
Angle of load to grain θ	45°	$67\frac{1}{2}°$
Percentage of basic load based on θ	82%	73%
Connector capacity at angle θ		
Dead load L	4,620	4,120
Dead load + snow load L_1	5,310	4,730
Joint stress		
Dead load P	4,580	3,360
Dead load + snow load P	10,130	7,420
Connectors required		
P/L	0.99	0.82
P_1/L_1	1.91	1.55
Connectors used	2	2
Capacity developed	95.5%	77.5%
Edge distance		
Compression side of chord	$3\frac{3}{4}$ in.	$3\frac{3}{4}$ in.
V_2	$2\frac{3}{4}$ in.	$2\frac{3}{4}$ in.
D_3	$2\frac{3}{4}$ in.	$2\frac{3}{4}$ in.
End distance		
D_3 $95.5 \times 82 = 78\%$	5 in.	
V_2 $77.5 \times 73 = 56.5\%$		$3\frac{1}{4}$ in.
Width of chord	$3\frac{3}{4} + 3\frac{3}{4} = 7\frac{1}{2}$ in.	

Connector Data and Loads	*D_3 to Chord*	*V_2 to Chord*
Width of V_2	$2\frac{3}{4} + 2\frac{3}{4} = 5\frac{1}{2}$ in.	
Width of D_3	$2\frac{3}{4} + 2\frac{3}{4} = 5\frac{1}{2}$ in.	
Size of members		
Chord $L_1 - L_2$	2—3 in. by 8 in.	
V_2	1—3 in. by 6 in.	
D_3	2—2 in. by 6 in.	

The size of the members must be checked for the stress in each member. The maximum stress in the lower chord is 34,450 lb. tension.

Required area = 34,450/1670 = 20.6 sq. in.

Two 3 by 8 in. pieces have an area of 39.38 sq. in. With connectors in both faces of each member, the net cross-sectional area becomes 27.94 sq. in. and two 3 by 8 in. pieces will satisfy.

The maximum stress in the diagonal tension member is 10,130 lb.

Required area = 10,130/1670 = 6.08 sq. in.

The net cross-sectional area of two 2 by 6 in. pieces is 12.10 sq. in., which is ample.

The maximum stress in V_2 is -7420 lb., and the length of the member = 5 ft. 4⅝ in.

L/d ratio = 25.

Allowable working stress = 960 lb. per sq. in.

Required area = 7420/960 = 7.73 sq. in.

A 3 by 6 in. member has an area of 14.77 sq. in.

In the heel joint, member U_0 transmits the horizontal component of its load through the wood splices to member L_0. This component acts at an angle of 22° in member U_0, and the number of rings between the top chord and the splice plates is determined by their capacity at this angle. The thickness of the top chord is made the same as the bottom chord so that the splice plates will be flat on the faces of both chords.

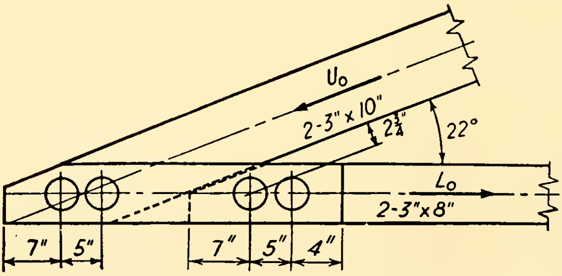


FIG. 95. Heel joint for 50-ft. Fink truss.

Connector Data and Loads	*Splice Plate to U_0*	*Splice Plate to L_0*
Size and type	4-in. split ring	4-in. split ring
Number of faces	2	2
Thickness of piece	$2\frac{5}{8}$ in.	$2\frac{5}{8}$ in.
Species group	*B*	*B*
Moisture content condition	Dry	Dry
Basic design load	5,635	5,635
Angle of load to grain θ	22°	0°
Percentage of basic load based on θ	94%	100%
Connector capacity at angle θ		
Dead load L	5,300	5,635
Dead load + snow load, L_1	6,100	6,480
Joint stress		
Dead load P	15,600	15,600
Dead load + snow load P_1	34,500	34,450
Connectors required		
P/L	2.96	2.78
P_1/L_1	5.65	5.32
Connectors used	8	8
Capacity developed	70.7%	66.5%
Spacing required	$4\frac{7}{8}$ in.	5 in.
Spacing used	5 in.	5 in.
Edge distance		
U_0	$2\frac{3}{4}$ in.	$2\frac{3}{4}$ in.
L_0	$2\frac{3}{4}$ in.	$2\frac{3}{4}$ in.
End distance		
Splice plates	$3\frac{1}{2}$ in.	4 in.
Chord L_0		4 in.
Chord U_0	4 in.	
Size of members		
L_0	2—3 in. by 8 in.	
U_0	2—3 in. by 10 in.	

The size of the lower chord L_0 has been determined in the previous joint design. The size of the upper chord U_0 depends on the load and on the L/D ratio. U_0 is designed as a spaced column with a length of 6 ft. $8\frac{3}{4}$ in. and a least dimension of $2\frac{5}{8}$ in.

$$\frac{L}{D}\text{ ratio} = 31$$

$$K = 0.702\sqrt{\frac{2.5 \times 1{,}600{,}000}{1380}} = 37.8$$

The column is in the intermediate class, and

$$\text{The allowable working stress} = 1380\left[1 - \frac{1}{3}\left(\frac{80.75}{37.8 \times 2.625}\right)^4\right]$$

$$= 1180 \text{ lb. per sq. in.}$$

$$\text{Required area} = \frac{37{,}030}{1180} = 31.2 \text{ sq. in.}$$

Two 3 by 8 in. pieces would satisfy, but in order to provide the necessary edge distances and spacing two 3 by 10 in. pieces are required.

At the peak joint it is necessary to use a filler of the same thickness as the compression web members to transmit the horizontal component to the other half of the truss. This component acts on member U_3 at an angle of 22°, and the load in member D_4 acts on U_3 at 22°.

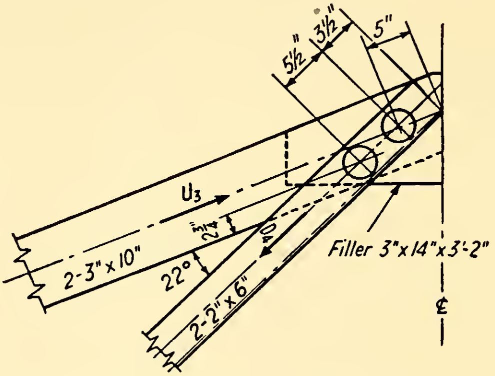


FIG. 96. Peak joint for 50-ft. Fink truss.

Connector Data and Loads	*Splice Plate to U_3*	*D_4 to U_3*
Size and type	4-in. split ring	4-in. split ring
Number of faces	2	2
Thickness of piece	$2\frac{5}{8}$ in.	$2\frac{5}{8}$ in.
Species group	*B*	*B*
Moisture content condition	Dry	Dry
Basic design load	5,635	5,635
Angle of load to grain θ	22°	22°
Percentage of basic load based on θ	94%	94%
Connector capacity at angle θ		
Dead load L	5,300	5,300
Dead load + snow load L_1	6,100	6,100

Connector Data and Loads	*Splice Plate to U_3*	*D_4 to U_3*
Joint stress		
Dead load P	8,750	6,820
Dead load + snow load P_1	19,320	15,060
Connectors required		
P/L	1.65	1.29
P_1/L_1	3.17	2.47
Connectors used	4	4
Capacity developed	80%	62%
Spacing required	$\theta = 22°$	22°
	$\phi = 22°$	22°
	$R = 7.15$ in.	7.15 in.
	Reduced $R = 5\frac{1}{2}$ in.	5 in.
Spacing used	$5\frac{1}{2}$ in.	$5\frac{1}{2}$ in.
Edge distance		
U_3	$2\frac{3}{4}$ in.	$2\frac{3}{4}$ in.
D_4	$2\frac{3}{4}$ in.	$2\frac{3}{4}$ in.
End distance		
U_3		$4\frac{3}{4}$ in.
D_4 $94 \times 62 = 58\%$		$3\frac{1}{2}$ in.

The size of the members has been determined in the design of the other joints.

The end distance required in member U_3 locates the gage line for the end connectors. In order to maintain the required edge distances for all members it is necessary to offset the center line of D_4 from the panel point. This slight eccentricity may be disregarded in design.

139. Design of an 80-Ft. Fink Truss. Figure 97 illustrates one-half of an 80-ft. Fink truss. The loads in the members are based on a dead load

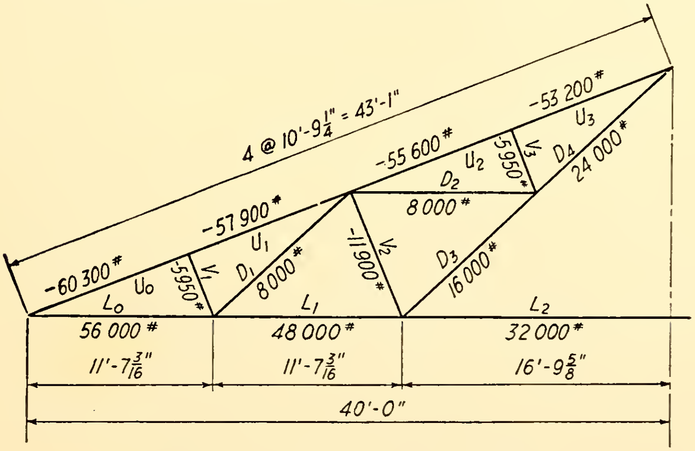

4 @ 10'-9¼" = 43'-1"
-53 200#
U3
-55 600#
U2
D2
-5950#
V3
D4
24 000#
8 000#
-57 900#
U1
D1
8000#
-11900#
V2
D3
16 000#
-60 300#
U0
-5950#
V1
L0
L1
L2
56 000#
48 000#
32 000#
11'-7 3/16"
11'-7 3/16"
16'-9 5/8"
40'-0"

FIG. 97. One-half of 80-ft. Fink truss.

of 40 lb. per sq. ft. of horizontal surface with the trusses 16 ft. on centers. Use the same grade of lumber and working stresses as in the example on p. 176.

Design of Members

Top chord: Length = 10 ft. 9¼ in. Select a timber 4 in. in nominal thickness.

$$\frac{L}{d} = \frac{129.25}{3.625} = 35.7$$

From the spaced column formula,

$$\frac{P}{A} = c\left[1 - \frac{1}{3}\left(\frac{L}{K_2 d}\right)^4\right]$$

the working stress becomes 960 lb. per sq. in.

$$\text{Area required} = \frac{60{,}300}{960} = 62.8 \text{ sq. in.}$$

Two 3 by 12 in. timbers with a cross-sectional area of 60.38 sq. in. will satisfy the requirements.

Bottom chord:

$$\text{Required area} = \frac{56{,}000}{1450} = 38.6 \text{ sq. in.}$$

Two 3 by 8 in. members with a cross-sectional area of 39.38 sq. in. will satisfy the requirements, but the joint between L_1 and L_2 will probably require a larger width.

Web members: Select a member 4 in. in nominal thickness for the web members in compression. These will be designed as solid columns.

V_2 has a length of 103.375 in.

$$\frac{L}{d} = \frac{103.375}{3.625} = 28.5$$

$$K = 0.702\sqrt{\frac{1{,}600{,}000}{1200}} = 25.6$$

$$\frac{P}{A} = \frac{0.329E}{\left(\frac{L}{d}\right)^2} = \frac{0.329 \times 1{,}600{,}000}{(28.5)^2} = 650 \text{ lb. per sq. in.}$$

$$\text{Required area} = \frac{11{,}900}{650} = 18.3 \text{ sq. in.}$$

A 3 by 8 in. member with a cross-sectional area of 19.69 sq. in. will be satisfactory. However, the lower chord joint may require a larger width.

Using a 3-in. nominal or 2⅝-in. actual thickness for compression members V_1 and V_3, which corresponds in thickness to V_2, the L/d ratio becomes $51.688/2.625 = 19.7$. Since this L/d ratio is greater than eleven but less than K, the formula for intermediate columns is used in calculating the allowable working stress.

$$\frac{P}{A} = 1200\left[1 - \frac{1}{3}\left(\frac{19.7}{25.6}\right)^4\right] = 1060 \text{ lb. per sq. in.}$$

$$\text{Required area} = \frac{5950}{1060} = 5.72 \text{ sq. in.}$$

If 4-in. split rings are used the minimum width permitted is 5½ in. This requires a 3 by 6 in. member with a cross-sectional area of 14.77 sq. in.

The maximum load for the diagonal tension members is in member D_4.

$$\text{Required area} = \frac{24{,}000}{1450} = 16.53 \text{ sq. in.}$$

Two 2 by 8 in. members with a cross-sectional area of 20.39 sq. in. are required.

For members D_1 and D_2 two 2 by 6 in. pieces will satisfy the requirements.

Design of Joints

Heel joint:

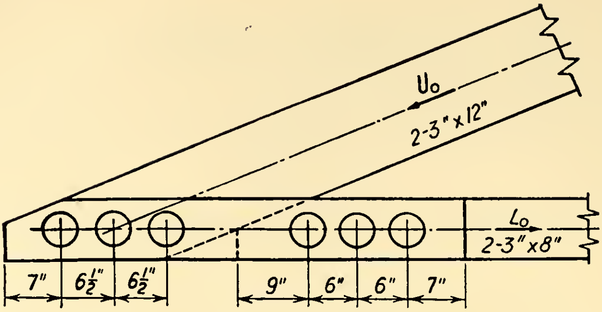


Fig. 98. Heel joint for 80-ft. Fink truss.

	Member U_0 and Splice Plates	*Member L_0 and Splice Plates*
Split-ring size	4 in.	4 in.
Number of contact faces for each ring	2	2
Thickness of piece	2⅝ in.	2⅝ in.
Angle of load to grain	22°	0°
Standard design load for one connector	5,300 lb.	5,635 lb.
Load on joint	56,000 lb.	56,000 lb.
Connectors required	10.60	9.95
Connectors used	12	12
Percentage of capacity developed	88%	83%
Standard spacing	7½ in.	9 in.
Reduced spacing	6½ in.	6 in.
Edge distance required	2¾ in.	2¾ in.
End distance, splice plates	7 in.	7 in.

End distance, U_0, 5½ in.

End distance, L_0, 9 in.

Lower chord joint:

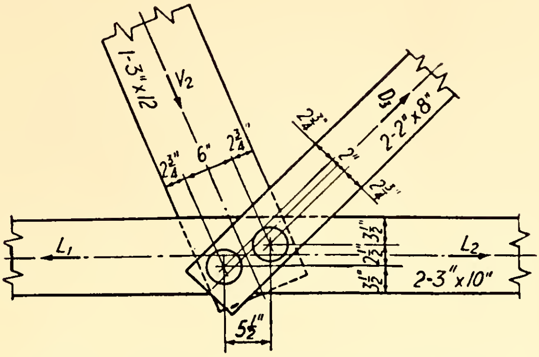


FIG. 99. Lower-chord joint for 80-ft. Fink truss.

	V_2 to Chord	*D_3 to Chord*
Split-ring size	4 in.	4 in.
Number of contact faces for each ring	2	2
Thickness of piece	$2\frac{5}{8}$ in.	$2\frac{5}{8}$ in.
Angle of load to grain	$67\frac{1}{2}°$	45°
Standard design load for one ring	4,120 lb.	4,620 lb.
Load on joint	11,900 lb.	16,000 lb.
Connectors required	2.9	3.46
Connectors used	4	4
Percentage of capacity developed	72.5%	86.5%
Standard edge distance, compression side of chord	$3\frac{3}{4}$ in.	$3\frac{3}{4}$ in.
Edge distance reduced	$2\frac{3}{4}$ in.	$2\frac{3}{4}$ in.
Spacing required parallel to grain	$5\frac{1}{2}$ in.	$5\frac{1}{2}$ in.

Width of chord, $2\frac{3}{4} + 2\frac{1}{2} + 2\frac{3}{4} = 8$ in. (use $9\frac{1}{2}$ in.)

Width of V_2, $2\frac{3}{4} + 6 + 2\frac{3}{4} = 11\frac{1}{2}$ in. (use $11\frac{1}{2}$ in.)

Width of D_3, $2\frac{3}{4} + 2 + 2\frac{3}{4} = 7\frac{1}{2}$ in. (use $7\frac{1}{2}$ in.)

End distance, V_2 in compression, $4\frac{1}{2}$ in.

End distance, D_3 in tension, $4\frac{1}{2}$ in.

Peak joint:

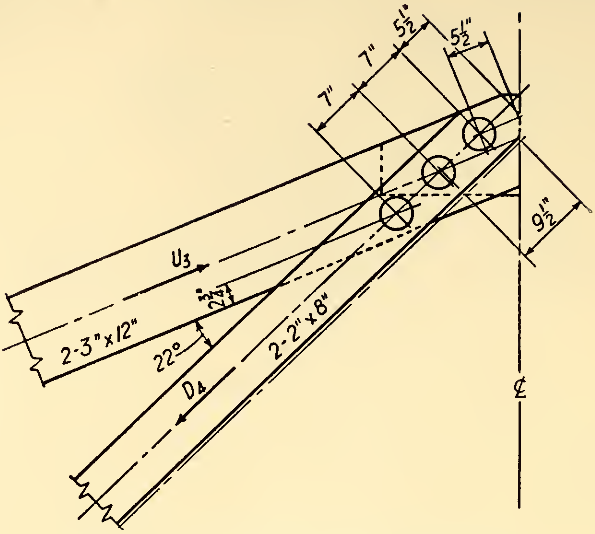
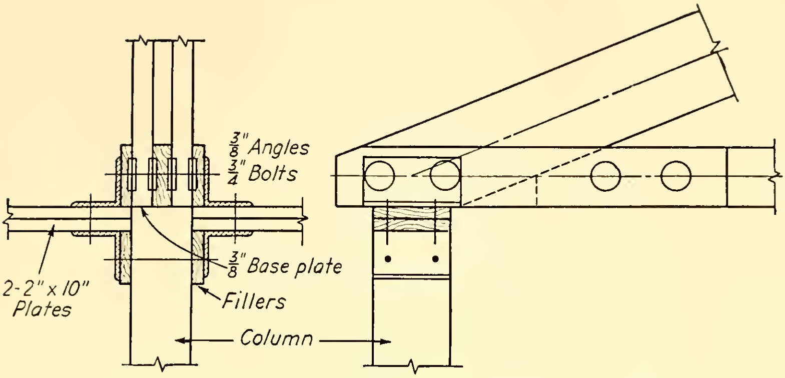


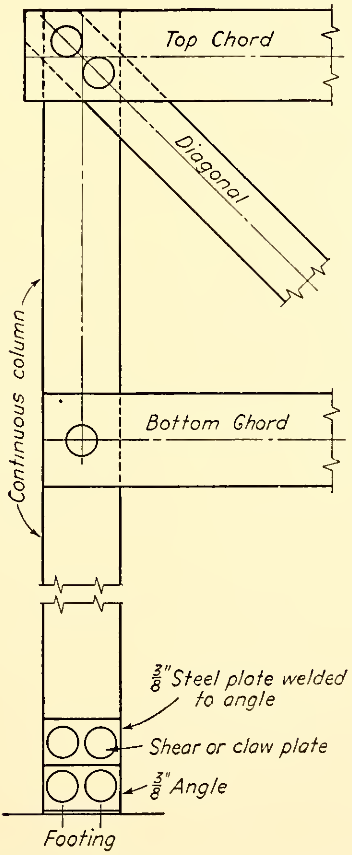
FIG. 100. Peak joint for 80-ft. Fink truss.

	D_4 *to* U_3	U_3 *to Splice Plates*
Split-ring size	4 in.	4 in.
Number of contact faces for each ring	2	2
Thickness of piece	$2\frac{5}{8}$ in.	$2\frac{5}{8}$ in.
Angle of load to grain	22°	22°
Standard design load for one connector	5,300 lb.	5,300 lb.
Load on joint	24,000 lb.	32,000 lb.
Connectors required	4.52	6.04
Connectors used	6	8
Percentage of capacity developed	75.5%	75.5%
Standard spacing required parallel to grain in D_4	9 in.	9 in.
Spacing used	7 in.	7 in.
End distance, D_4, $5\frac{1}{2}$ in.		
End distance, U_3, $5\frac{1}{2}$ in.		
Edge distance required	$2\frac{3}{4}$ in.	$2\frac{3}{4}$ in.



3/8" Angles
3/4" Bolts
3/8" Base plate
Fillers
2-2" x 10" Plates
Column

FIG. 101. Method of anchoring wood trusses to wood columns.



Top Chord
Diagonal
Continuous column
Bottom Chord
3/8" Steel plate welded to angle
Shear or claw plate
3/8" Angle
Footing

FIG. 102. Built-up columns made an integral part of the truss.

140. Column and Wall Attachments. The method of anchoring trusses to wood columns can be best explained by means of Fig. 101. It is often desirable to use a built-up column and have it act as an integral part of the truss. This type of construction is illustrated in Fig. 102. Attaching a wood truss to a brick or concrete wall requires the use of anchor bolts that are fastened to the angles on the sides of the bottom chord.

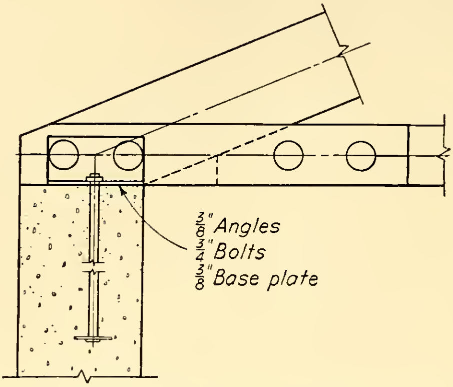


FIG. 103. Method of anchoring wood trusses to masonry walls.

141. Camber. Connectors have been used in the design of the trusses in this chapter, but the possibility of using bolts only in many joints or bolts in addition to the connectors should not be overlooked. Before connectors were used in this country it was common practice to design trusses with bolted joints, using steel rods for the tension members. This made it possible to put the camber in the truss after it was erected by merely tightening the tension rods. Moreover, wood will sag in time, and the above method of construction permitted the truss to be raised to the proper camber after several years of service. The usual cambers were between 2 and 4 in.

The steel rods at the center of the Fink trusses designed in this chapter have somewhat the same purpose of raising the bottom chord or at least in keeping it from sagging. However, in most trusses designed with connectors and all wood members, it is necessary to fabricate and erect the truss with the camber already provided. The amount of camber depends upon the size of the truss, but a good estimate is between 1 in. and 6 in. for spans between 30 and 100 ft. The following table, which lists the total slip for joints with connectors, may be helpful in estimating the movement in the joints of wood trusses.

Table 38. Deformation, in Inches, for Joints Using Connectors

Connector	*Size*	*Bolt*	*Total Deformation* Load Parallel to Grain	Load Perpendicular to Grain
Split rings	$2\frac{1}{2}$	$\frac{1}{2}$	0.040	0.050
	4	$\frac{3}{4}$	0.045	0.055
	6	$\frac{3}{4}$	0.050	0.060
Toothed rings	2	$\frac{1}{2}$	0.004	0.006
	$2\frac{5}{8}$	$\frac{5}{8}$	0.006	0.008
	$3\frac{3}{8}$	$\frac{3}{4}$	0.008	0.010
	4	$\frac{3}{4}$	0.010	0.012
Claw plates	$2\frac{5}{8}$	$\frac{1}{2}$	0.015	0.023
	$3\frac{1}{8}$	$\frac{1}{2}$	0.038	0.045
	4	$\frac{3}{4}$	0.038	0.045
Shear plates	$2\frac{5}{8}$	$\frac{3}{4}$	0.073	0.100
	4	$\frac{3}{4}$	0.098	0.102
	4	$\frac{7}{8}$	0.050	0.090
Spike grids, flat		$\frac{3}{4}$	0.012	0.013
		1	0.015	0.021
single curve		$\frac{3}{4}$	0.011	0.020
		1	0.018	0.025
double curve		$\frac{3}{4}$	0.011	0.020
		1	0.018	0.025

A formula developed by the Timber Engineering Company for calculating the camber in trusses built with connectors is as follows:

$$\Delta = K_1 \frac{L^3}{H} + K_2 \frac{L^2}{H}$$

where Δ = recommended camber in inches at center of truss.
L = span of truss in feet.
H = height of truss in feet at center.
K_1 = 0.000032 for any type of truss.
K_2 = 0.0028 for flat and pitched trusses, or 0.00063 for bowstring trusses; that is, trusses without splices in the upper chord.

PROBLEMS

1. If the bottom chord of a Fink truss consists of two 3 by 8 in. pieces and is spliced at the heel joint with the top chord of two 3 by 14 in. pieces entering at an angle of 22°, how many connectors are required to carry a load of 43,000 lb. in the top chord? Sketch the joint showing the required spacing and the end and edge distances for the connectors.

2. The bottom chord of a truss consists of two 3 by 8 in. members in tension with loads on each side of a panel point of 34,300 lb. and 22,000 lb. Two diagonal 2 by

6 in. tension members enter the joint at 45° to the chord and carry a load of 11,500 lb. A single 3 by 6 in. compression member carrying a load of 8400 lb. enters the joint at 67½° to the chord. What size and how many connectors are required? Detail the joint.

3. Design and detail joint L_0–L_1, shown in Fig. 90.

4. Design and detail the top and bottom chord splices for the truss illustrated in Fig. 90.

5. For the truss shown in Fig. 97, design and detail joint U_1–U_2.

6. For the same truss as in the previous problem, design and detail joint D_3–D_4.

7. Design and detail a 40-ft. Pratt truss with eight panels 5 ft. long. The end height of the truss is 4 ft. and the center height is 5 ft. 4 in. The trusses are spaced 16 ft. on centers. Consider the weight of the truss, and assume the weight of purlins, sheathing, and roofing to be 10 lb. per sq. ft. The snow load is 10 lb. per sq. ft. and the wind load is 15 lb. per sq. ft. Use select structural Douglas fir.

CHAPTER VIII

TIMBER DECKS AND BRIDGES

142. General. Timber bridges may be divided into two general classes, namely, the trestle and the truss bridge. A trestle bridge usually consists of a number of pile bents, and the simplest form is to use the pile as both post and footing. However, in many trestle bridges the piles are driven and capped at the ground line, and a frame bent is built on top of the pile foundation. In either type of construction it is necessary to provide bracing to keep the structure from swaying.

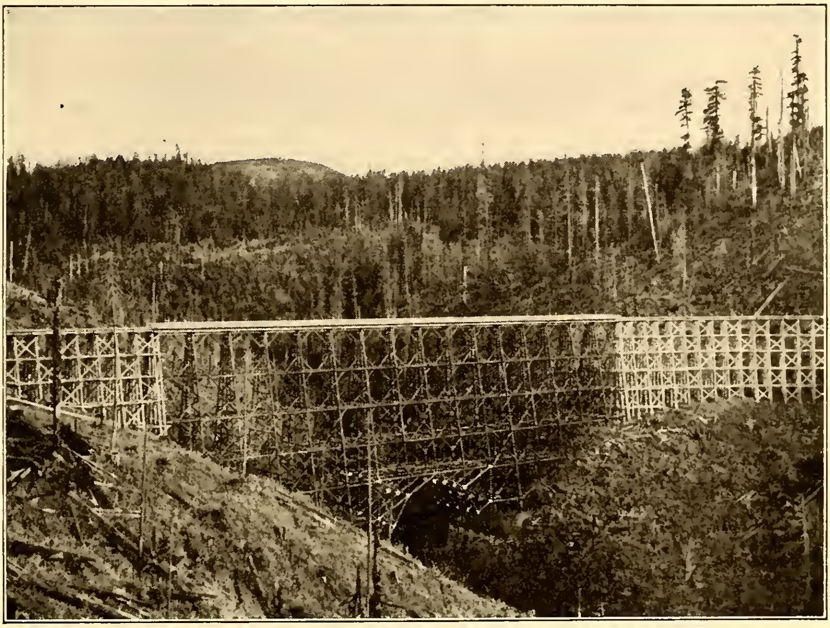


Fig. 104. Baird Creek Canyon Bridge constructed for Weyerhauser Timber Company. The bridge has an over-all length of 1130 ft. and a rise of 235 ft.

The truss bridge in its simplest form is the A-frame or King post truss and is suitable for spans up to about 40 ft. The Queen post truss is an adaptation of the A-frame with one panel of horizontal top chord added between the end posts. These have been used for spans up to 70 ft. A number of large timber truss bridges have been built recently with spans of 150 to 210 ft. All these have utilized the connector system of construction, which provides economies and spans that have heretofore been impossible.

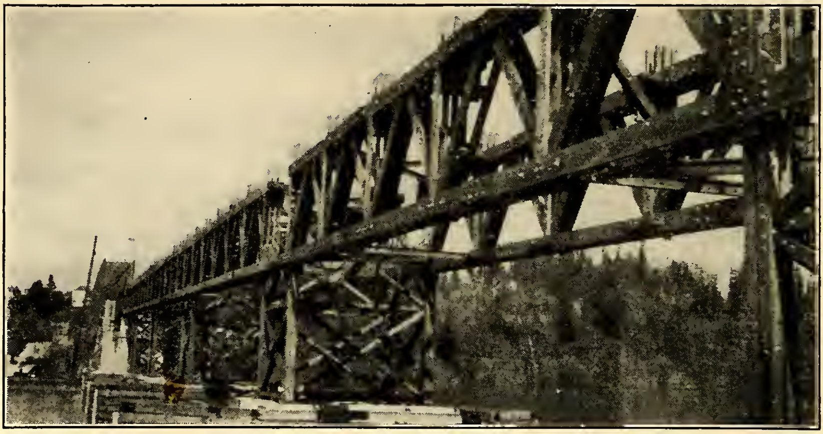


Fig. 105. Puget Island-Cathlamet, Washington, Bridge over Columbia River. Bridge is 2400 ft. long and is built with Wolmanized Douglas fir and Teco connectors. Fabricated by Timber Structures, Inc., and built by Washington Department of Highways.

In both types of bridges, the number should be given a pressure treatment of an approved preservative in accordance with current practice for the condition of exposure of the structure. Moreover, the exposed

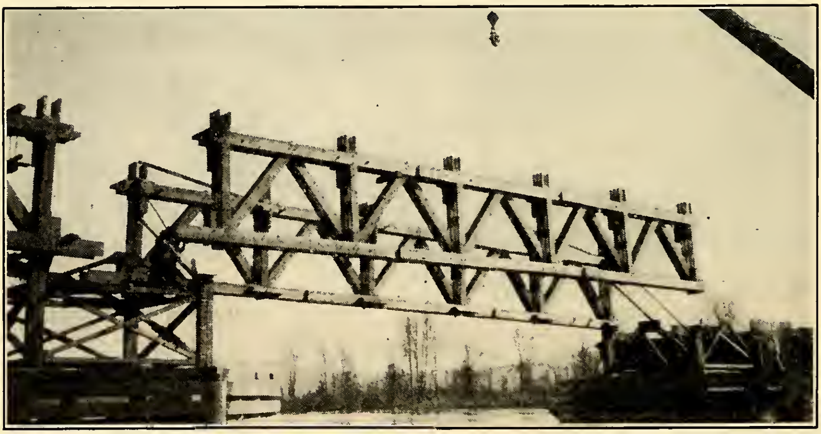


Fig. 106. A 100-ft. truss for the Puget Island-Cathlamet, Washington, Bridge being swung into place by a large crane on a barge.

portions of treated pieces that have been cut in the field should be given an adequate treatment to make them as durable as the remaining portion. Piles that have been cut off are usually protected by several coats of hot creosote and pitch and a cap of galvanized iron. Pre-framing and boring of timbers at the treating plant, before the preservative is applied, is recommended. When it is necessary to bore a treated timber, the hole should be thoroughly soaked with hot preservative.

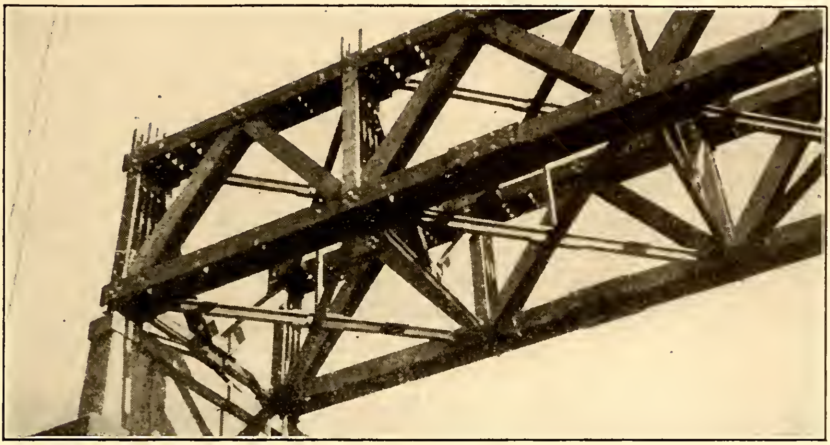


FIG. 107. Close up view of Wolmanized Douglas fir trusses for Puget Island-Cathlamet, Washington, Bridge.

Bridge decks are usually made up of a stringer system with a plank or laminated floor or with continuous timber-concrete composite assemblies. This latter form, which combines the ability of wood to resist tensile stresses and concrete to resist compressive stresses, has proved successful. The timbers are placed on edge with adjacent timbers varying in depth by about 2 in. This forms longitudinal grooves into which metal plates are driven to provide a shear connection between the timber on the bottom and the concrete mat on the top of the assembly.

143. Factors Affecting Design. The design of any structure depends upon three main factors: adequacy, economy, and aesthetics. By adequacy is meant that the structure should be so designed that it will resist the loads and forces to which it will be subjected with a reasonable factor of safety. Moreover, the structure should be adequate to perform the function for which it is intended. A bridge is part of a railway or highway system and, as such, should be designed in relation to the primary project of which it is probably only a small part. The existence of numerous timber trestles in railroad construction and truss and deck bridges along our highways seems to be proof that timber bridges can be designed to adequately carry the loads ordinarily encountered in this type of construction.

The economy of any design depends upon the initial cost, maintenance cost, and permanency of the structure. The relative economy of different types of structures is not determined by the initial costs alone but rather by comparing the annual cost which includes a portion of the initial cost, the maintenance charges, and an annuity which will produce a sum sufficient to reproduce the structure. The items of first cost and interest charges are known quantities or can be estimated for any type of bridge. Consequently, maintenance costs and the permanency of a structure are the two considerations involved in rating one design more advantageous than another. Maintenance cost depends upon the durability of the material. Statistics are available which show that the average life of creosoted railway crossties is not less than 30 years, that exposed creosoted timber piles have an average life of 35 to 40 years, and stringers and decking in ballasted deck bridges, 40 years. In the ballasted deck bridges used in railway structures, the deck material is protected from exposure and therefore is not subject to mechanical wear. This type of structure may be considered equivalent to highway bridges having a suitable wearing surface on top of the timber deck.

The maintenance cost for any structure is not readily estimated without actual service records and costs of similar structures. Timber decks with a suitable wearing surface usually require maintenance of the wearing surface before it is necessary to repair or replace any of the beams or stringers. The timber-concrete composite deck has the best rating from a maintenance standpoint. Table 39 gives the total amount expended for maintenance and repairs for each year of service for 129 ballasted deck creosoted bridges. From the figures it is possible to calculate the cost per linear foot per year of age.

An estimate of the permanence of any structure requires consideration of the fact that the structure may become obsolete or inadequate before the end of its expected life. Consequently, in estimating the life of a structure, the factor of permanence should be considered with caution. While it is true that the expected life may be measured in centuries, a study of the histories of many bridges will show obsolescence from anywhere between 10 and 50 years.

Many timber bridges are not very beautiful structures, but there seems to be no reason why properly designed timber bridges cannot be made as pleasing in appearance as other structures. One of the main requirements of aesthetics in design is to produce a structure that will be in harmony with its surroundings. Proper treatment of balustrades and the increased use of built-up members and composite decks can add much to the appearance of timber bridges.

Table 39. Maintenance and Repair Costs for 129 Ballasted Deck Creosoted Timber Railway Bridges in Louisiana

["Timber Bridge Structures," by J. F. Seiler, *Am. Road Builders' Assoc.*, Bull. 43-A, 1935. From Office of Chief of Engineers, U. S. Army]

TOTAL COSTS FOR YEAR OF SERVICE SHOWN

Bridge Number	1	2	3	4	5	6	7	8
1 to 20 Inc.	$22.09	$101.66	$ 27.56	$ 48.33	$103.69	$232.11	$292.45	$ 80.56
21 to 43 Inc.	35.98	58.54	177.98	284.17	405.53	38.55	769.36	1,311.14
44 to 64 Inc.	....	136.19	41.52	133.27	403.46	170.17	488.75	191.53
65 to 85 Inc.	....	45.06	....	156.79	107.00	76.80	184.63	297.78
86 to 107 Inc.	....	....	....	....	95.34	....	740.74	131.62
108 to 129 Inc.	....	....	....	....	205.15	....	....	....
Totals	$58.07	$341.45	$247.06	$622.56	$1,320.17	$517.63	$2,475.93	$2,012.63
Footage in Service	15,571	15,571	15,571	15,455	15,455	15,252	15,162	15,121

Bridge Number	9	10	11	12	13	14	15	16
1 to 20 Inc.	$1,100.22	$ 17.57	$ 37.38	$ 844.13	$ 868.11	$403.82	$1,005.73	$ 301.16
21 to 43 Inc.	242.98	4,393.72	1,450.76	1,299.25	1,365.24	542.88	410.95	1,945.40
44 to 64 Inc.	131.69	125.48	40.55	162.43	36.53	517.22	48.34	163.48
65 to 85 Inc.	99.86	185.82	65.70	46.98	205.18	310.31	211.11	229.70
86 to 107 Inc.	215.93	44.04	432.14	30.79	230.71	305.65	129.18	313.97
108 to 129 Inc.	....	552.52	429.46	83.45	....	367.20	62.46	....
Totals	$1,790.68	$5,319.15	$2,455.59	$2,467.03	$2,705.77	$2,447.08	$1,867.77	$2,953.71
Footage in Service	15,058	14,408	14,381	12,189	11,619	10,406	9,368	6,894

Bridge Number	17	18	19	20	21	22	23	24
1 to 20 Inc.	$ 240.84	$ 622.44	$131.92	$278.58	$1,577.38	$ 98.48	$63.46	....
21 to 43 Inc.	988.49	333.94	87.57	44.68	48.45	106.32	77.85	....
44 to 64 Inc.	285.92	169.80	48.09	49.53	174.79	74.02	....	....
65 to 85 Inc.	232.06	372.28	....	....	330.26	....	....	....
86 to 107 Inc.	537.99	91.88	....	....	104.29	....	....	....
108 to 129 Inc.	....	....	....	....	....	....	....	....
Totals	$2,285.30	$1,590.34	$267.58	$372.79	$2,235.17	$278.82	$141.31	....
Footage in Service	5,892	4,005	1,698	996	914	805	709	615

A.A.S.H.O. SPECIFICATIONS

The American Association of State Highway Officials has set forth specifications for the selection of material and the design of highway bridges in its Standard Specifications for Highway Bridges. The following articles are taken from or based on these specifications.

144. Timber Piles. Timber piles which will be below water level at all times may be of any species of wood which will satisfactorily withstand driving.

In untreated piling for use in exposed work, the diameter of the heartwood shall be not less than eight-tenths of the required diameter of the pile.

All wood piling shall be cut from sound and live trees, preferably during the winter season. They shall contain no unsound knots. Sound knots will be permitted, provided the diameter of the knot does not exceed 4 in. or one-third of the diameter of the stick at the point where it occurs. Any defect or combination of defects which will impair the strength of the pile more than the maximum allowable knot shall not be permitted. The butts shall be sawed square, and the tips shall be sawed square or tapered to a point not less than 4 in. in diameter as directed by the engineer.

Unless otherwise specified, all piles shall be peeled by removing all of the rough bark and at least 80 per cent of the inner bark. No strip of inner bark remaining on the stick shall be over ¾ in. wide or over 8 in. long, and there shall be at least 1 in. of clean wood surface between any two such strips. Not less than 80 per cent of the surface of any circumference shall be clean wood.

Piles shall be cut above the ground swell and shall taper from butt to tip. A line drawn from the center of the tip to the center of the butt shall not fall outside of the center of the pile at any point more than 1 per cent of the length of the pile. In short bends, the distance from the center of the pile to a line stretched from the center of the pile above the bend to the center of the pile below the bend shall not exceed 4 per cent of the length of the bend or a maximum of 2½ in. All knots shall be trimmed close to the body of the pile.

Round piles shall have a minimum diameter at the tip, measured under the bark as follows:

Length of Pile	*Tip Diameter*
Less than 40 ft.	8 in.
40 to 60 ft.	7 in.
Over 60 ft.	6 in.

The minimum diameter of piles at a section 3 ft. from the butt, measured under the bark, shall be as follows:

	Diameter in Inches	
Length of Pile	Douglas Fir, Southern Yellow Pine	All Other Species
20 ft. and under	11	11
21 to 30 ft.	12	12
31 to 40 ft.	12	13
Over 40 ft.	13	14

The diameter of the pile at the butt shall not exceed 20 in. The diameter of a pile in cases where the tree is not exactly round shall be determined either by measuring the circumference and dividing the number of inches by 3.14, or by taking the average of the maximum and minimum diameters at the location specified.

When required, the size and number of piles shall be determined by actual loading tests. In general, these tests shall consist of the application of a test load placed upon a suitable platform supported by the pile, with suitable apparatus for accurately measuring the test load and the settlement of the pile under each increment of load.

In lieu thereof hydraulic jacks with suitable yokes and pressure gages may be used.

The safe allowable load shall be considered as 50 per cent of that load which, after a continuous application of 48 hours, produces a permanent settlement not greater than ¼ in. measured at the top of the pile. This maximum settlement shall not be increased by a continuous application of the test load for a period of 60 hr. or longer. At least one pile for each group of 100 piles shall be tested.

In the absence of loading tests, the safe bearing values for timber piles shall be determined by the following formulas:

$$P = \frac{2WH}{S + 1.0} \text{ for gravity hammers}$$

$$P = \frac{2WH}{S + 0.1} \text{ for single-acting steam hammers}$$

$$P = \frac{2H(W + Ap)}{S + 0.1} \text{ for double-acting steam hammers}$$

where P = safe bearing power in pounds.
W = weight of striking parts of hammer in pounds.
H = height of fall in feet.
A = area of piston in square inches.
p = steam pressure in pounds per square inch at the hammer.
S = the average penetration in inches per blow for the last 5 to 10 blows for gravity hammers and the last 10 to 20 blows for steam hammers.

The above formulas are applicable only when

(*a*) the hammer has a free fall,
(*b*) the head of the pile is not broomed or crushed,
(*c*) the penetration is reasonably quick and uniform,
(*d*) there is no sensible bounce after the blow,
(*e*) a follower is not used.

Twice the height of the bounce shall be deducted from H to determine its value in the formula.

Unless otherwise ordered by the engineer, timber piling shall be driven to the bearing value given on the plans or in the supplemental specifications. If bear-

Table 40. Structural Grades for Highway Bridge Construction

[Standard Specifications for Highway Bridges, A.A.S.H.O., 1941]

STRUCTURAL PURPOSE	SIZE OF MEMBER	STANDARD GRADE [1]
(*a*) Truss members, tension Floor beams Stringers Other floor members	5″ x 8″ and larger	1800#f, or 1600#f, or 1400#f structural beams and stringers
(*b*) Caps Posts, bridge and guard rail Sills Mud sills Nailing strips Truss members, compression Timbers (culverts)	6″ x 6″ and larger	1200#c or 1100#c structural posts and timbers
(*c*) Joist Decking, wearing Other floor members Rails Rail posts Nailing strips Truss members, compression and tension Guard rail	4″ and thinner	1800#f, 1600#f, or 1400#f structural joist and plank
(*d*) Wheel and felloe guards	6″ x 6″ and larger	1100#c structural posts and timbers
(*e*) Sub-decking, flat Sub-decking, laminated Bracing, sway, sash, and longitudinal Girts Bulkhead plank Scupper blocks Cleats Grillage	4″ and thinner	1200#f or 1100#f joist and plank
(*f*) Cross-bridging Sidewalk Firestops	2″ and 3″ thick	No. 1 dimension
(*g*) Truss housing Inside sheathing	1″ and 1¼″ thick	D select finish, or No. 1 boards

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[1] For temporary structures which are for use only during erection or for emergency use, the grades of 1200#f or 1100#c may be substituted for 1800#f, 1600#f, 1400#f, or 1200#c where specified above; No. 1 Dimension for 1200#f and 1100#f, and No. 1 timbers for 1100#f.

ing values are not given, timber piling shall be driven to a minimum value of 20 tons.

In case water jets are used in connection with the driving, the bearing power shall be determined by the above formulas from the results of driving after the jets have been withdrawn, or a load test may be applied.

145. Structural Timber. The stress grades for structural timber conform to the principles of strength grading of the American Lumber Standards, Simplified Practice Recommendations, R-16, which represent complete grading rules suitable for both engineering design and lumber purchase and which incorporate all factors affecting the strength and utility of structural timbers and their commercial grading. Existing commercial grading rules of the regional lumber manufacturers' associations are in conformity, generally, with the principles of structural grading set forth by the A.A.S.H.O. and provide for grades of equal or higher working stresses.

146. Timber Grades for Particular Uses. For the various structural purposes the grades given in Table 40 may be used.

147. Distribution of Loads. For calculating the bending moments in longitudinal beams or stringers, a lateral but no longitudinal distribution of wheel load is permitted. The proportion of the wheel load for calculating the moments in interior stringers may be determined as follows:

$$W' = \frac{WS}{K} \text{ for one traffic lane}$$

$$W' = \frac{WS}{K'} \text{ for two traffic lanes}$$

W' = fraction of wheel load to each stringer
W = wheel load in pounds
S = average spacing of stringers in feet
K = 4.0 plank floors
4.5 strip floor 4 in. thick
5.0 strip floor 6 in. thick or thicker
K' = 3.75 plank floors
4.00 strip floor 4 in. thick
4.25 strip floor 6 in. thick or thicker

The live load supported by outside stringers is the reaction of the truck wheels, assuming that the flooring acts as a simple beam between stringers.

PILES

148. Purpose of Piles. Piles are used primarily for the following reasons:

1. To transmit the loads from a structure through material having little or no bearing capacity to one having adequate bearing capacity.
2. To compact or densify a loose material.
3. To protect wharves, docks, and other marine structures.

149. Types of Piles. Piles may be classified in accordance with the manner in which they develop bearing capacity. The "friction pile" transfers its load to the adjacent soils by friction along its embedded length. The "point-bearing pile" transmits all of its load by point bearing on a firm stratum. Between these two types are piles combining both skin friction, over part of their length, and point bearing in developing their load capacity.

150. Loads. Piles may be required to carry a variety of loads, depending on their use and the type of structure they support. Piles are generally designed to carry axial loads acting downward, which consist of the dead and live loads of the structure supported by the piles. Downward axial loads can also be caused by the friction of the upper layers of earth when the lower layers consolidate.

Piles are often subjected to axial loads acting upward and tending to push the piles up vertically. Loads causing this condition are those due to wind on a tall structure, the pull of the cables on a suspension bridge, lateral and vertical loads on retaining walls, and other similar loads. The ability of a pile to resist loads of this type depends upon skin friction or the frictional resistance between the pile and the soil.

If a group of piles are subjected to a lateral force, the force may be resolved into a resisting couple tending to pull up one side of the group and lower the other.

151. Pile Driving. There are four methods commonly used in driving piles. These are the use of a gravity hammer, steam hammer, water jet, and hydraulic jack. The gravity hammer, usually referred to as the drop hammer, is merely a weight that is allowed to fall freely and hit the top of the pile. The weight of the hammer and the height of fall may be selected to give the desired kinetic energy per blow.

The steam hammers are of two types: single acting and double acting. The single-acting hammer utilizes steam pressure to raise the hammer which then falls by gravity. The double-acting steam hammer utilizes steam pressure on both the upward and the downward stroke.

Water jets are used to loosen sand or fine gravel in a compacted condition which makes driving difficult. After the use of a water jet the pile should be driven further to secure a firm bearing and to measure the final degree of penetration.

The hydraulic jack is generally used when the space is restricted. It is placed on top of the pile with resistance against a building or other structure over the pile.

The single-acting steam hammer has several advantages over the gravity hammer. These are more rapid blows, a shorter drop, and more accurate control. The double-acting steam hammer has a lighter weight for the same driving capacity at a given length of stroke and has a greater speed of driving than the drop hammer or single-acting steam hammer.

152. Pile Formulas. Numerous formulas have been derived for estimating the safe load which may be placed on a pile, and each formula will give a different result. Table 41 presents an example of the variation in the results calculated from a number of formulas and compared to actual static test loads. This table is taken from the Proceedings of the American Society of Civil Engineers, September, 1941. The table shows the ultimate loads by formulas and the actual ultimate static test loads at failure for eight timber piles. The piles were driven by a single-acting hammer into a cohesive soil.

Table 41. Comparison of Pile Formulas with Actual Test Loads at Failure, in Tons

Actual Loads at Failure (Tons)	Equation 19 (Redtenbacher)		Equation 22 (Hiley)		Equation 23 (Engineering News)		Equation 25 (Pacific Coast)		Equation 26 (Terzaghi)			
									$n^2 = 0.25$		$n^2 = 0.1$	
	Tons	%	Tons	%	Tons	%	Tons	%	Tons	%	Tons	%
Timber Piles: Single-Acting Steam Hammer and Cohesive Soils												
93.0	108.0	116	67.0	72	270.0	290	82.5	89	113.0	122	109.0	117
81.0	104.0	128	131.5	162	169.8	210	85.7	106	109.0	135	106.0	131
70.0	43.5	62	37.0	53	75.0	107	40.0	57	50.3	72	46.0	66
66.5	62.0	93	58.0	87	106.0	159	60.0	90	73.0	109	69.0	103
96.5	47.0	49	41.0	42	88.0	91	44.4	46	54.3	56	50.0	52
100.0	73.0	73	68.5	68	180.0	180	60.8	61	81.8	82	77.5	77
110.0	33.5	30	29.0	26	56.0	51	32.8	30	38.4	35	35.5	32
70.0	43.5	62	37.0	53	75.0	107	40.0	57	50.3	72	46.0	66

1. Also see "Pile Foundations and Pile Structures," Manual 27, American Society of Civil Engineers.

2. A load at failure is assumed to be the load that produced an increase in settlement disproportionate to the increase in pile load.

3. Percentage of test load at failure.

The formulas used in computing Table 41 are given in the A.S.C.E. Proceedings and are

Equation (22), Hiley, September, 1941, Proceedings
Equation (23), Eng. News, September, 1941, Proceedings
Equation (19), Redtenbacher, May, 1941, Proceedings
Equation (25), Pacific Coast, September, 1941, Proceedings
Equation (26), Terzaghi, September, 1941, Proceedings

In equations 19, 22, 25, and 26, R_d is the ultimate load. In equation 23, R is the safe load with a factor of safety of 6. In equations 19, 22, 25, and 26, h and s must be expressed in the same units, but, in equation 23, h is in feet and s is in inches.

Additional pile formulas (Hiley, simplified):

$$\text{For a drop hammer, } R = \frac{3Wh}{s+k} \times \frac{W}{W+P}$$

$$\text{For single-acting steam hammer, } R = \frac{3.6Wh}{s+k} \times \frac{W}{W+P}$$

$$\text{U. S. Navy (Bureau of Yards and Docks), } R = 2Wh$$

SYMBOLS IN PILE FORMULAS

W = weight of striking parts of hammer in pounds.
P = weight of pile as driven in pounds.
h = height of fall of hammer in feet or inches.
s = penetration of pile per blow in inches.
E = modulus of elasticity of pile as driven in pounds and inches.
A = cross-sectional area of pile as driven in square inches.
L = length of pile as driven in feet or inches.
n = coefficient of restitution.
e = efficiency of hammer in percentage.
R_d = ultimate bearing capacity of pile.
R = allowable static load on pile.

From Table 41 it is apparent that pile-driving formulas cannot always be relied upon and should be used with great discretion. Many uncertainties are involved in the determination of the bearing value of piles. The soil, by supporting the pile, must carry the foundation loads, and the properties and behavior of the different classes of soils are not very well known. Even with exploration tests at the site it is not possible to obtain an exact analysis of subsurface conditions. However, with the results of soil tests on soil taken at the site of any proposed foundation and with the uncertainties and limitations in pile formulas clearly kept in mind, a formula may still serve as a guide to the engineer in deciding the bearing value of an individual pile. For a complete discussion of pile-driving formulas see the A.S.C.E. Proceedings for September and October, 1941, and January and February, 1942.

153. Test Piles. For any large foundation it is customary to drive test piles. A test pile is one on which a known load is placed in order to determine its bearing capacity and settlement. The results of the test can be used to check the design or as a basis for selecting the proper type of pile and its bearing capacity. The usual method of applying the test load is to build a platform on top of the pile and load it with pig iron or sand.

The load increments should usually be from 5 to 10 tons each, and settlement readings should be taken after each load is applied. When the full design load has been applied, the pile should be kept under this load until there is no settlement for a 24-hour period. Then the load on the pile should be increased 50 per cent and this load maintained until the pile comes to rest for another 24-hour period. As the pile is then unloaded, the rebounds should be recorded.

Under no circumstances should the pile be loaded immediately after driving. The time lapse should be at least 48 hours.

154. Group Piling. It must be remembered that the results obtained from a test pile subjected to a static load will not present a true picture of the behavior of a large pile group. The bearing capacity of one pile cannot be multiplied by the number of piles in a group to obtain the safe bearing value of the foundation as a whole.

A method for deriving the bearing capacity of a group of piles has been suggested by Frank M. Masters in the A.S.C.E. Transactions, 1943. The value of a pile in a group is based on the value of a single pile less one sixteenth for each adjacent diagonal or row pile. A pile fully enclosed in a group, as the center pile in a square of nine piles, would have 50 per cent of the bearing capacity of a single pile.

The spacing of piles is usually taken as a minimum of 3 ft. for piles of ordinary size. In the design of spacing it is important to have a uniform bearing on all the piles. To accomplish this the resultant of the superimposed loads should coincide with the resultant of the resistance of the piles.

TIMBER TRESTLES

155. General. The decks of timber trestles are usually constructed of 2 by 4 in. or 2 by 6 in. material laid on edge with their length perpendicular to the direction of the traffic. This type of deck distributes the load over a greater area than the plank deck where the individual pieces are laid flat. Moreover, the edge loading conditions are less severe in the laminated deck because the pieces can be effectively tied together to act as one solid unit.

Span lengths up to 25 ft. are economical for timber trestles where all wood construction is used. However, beyond 25 ft. it is usually more economical to use rolled-steel beams for the stringer system. On span lengths over 15 ft. where wood stringers are used, it is customary to bolt 4 by 12 in. blocks between the stringers to act as bridging.

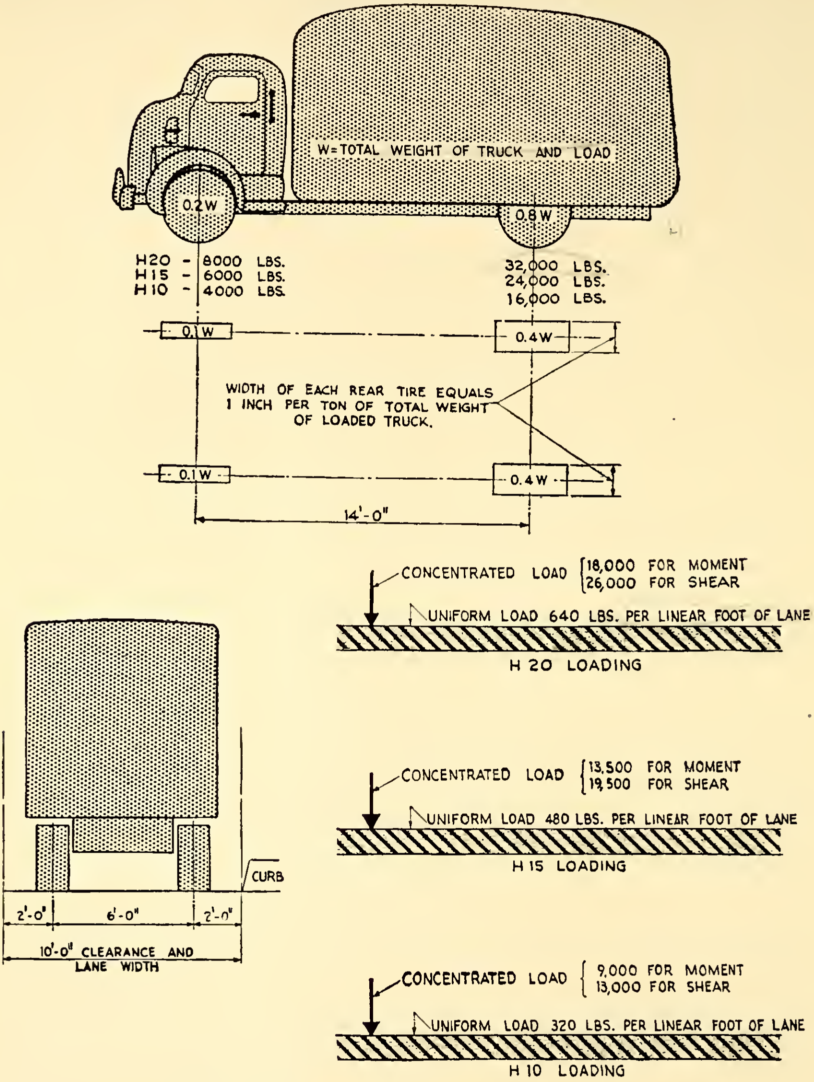

W=TOTAL WEIGHT OF TRUCK AND LOAD
0.2W
0.8W
H20 - 8000 LBS.
H15 - 6000 LBS.
H10 - 4000 LBS.
32,000 LBS.
24,000 LBS.
16,000 LBS.
0.1W
0.4W
WIDTH OF EACH REAR TIRE EQUALS 1 INCH PER TON OF TOTAL WEIGHT OF LOADED TRUCK.
0.1W
0.4W
14'-0"
CURB
2'-0"
6'-0"
2'-0"
10'-0" CLEARANCE AND LANE WIDTH
CONCENTRATED LOAD 18,000 FOR MOMENT 26,000 FOR SHEAR
UNIFORM LOAD 640 LBS. PER LINEAR FOOT OF LANE
H 20 LOADING
CONCENTRATED LOAD 13,500 FOR MOMENT 19,500 FOR SHEAR
UNIFORM LOAD 480 LBS. PER LINEAR FOOT OF LANE
H 15 LOADING
CONCENTRATED LOAD 9,000 FOR MOMENT 13,000 FOR SHEAR
UNIFORM LOAD 320 LBS. PER LINEAR FOOT OF LANE
H 10 LOADING

Fig. 108. Standard H truck and lane loadings.

156. Live Loads for Highway Bridges. In addition to the dead load or weight of a bridge, the live or moving loads that pass over the bridge must be provided for in design. The customary specifications are those of the American Association of State Highway Officials (A.A.S.H.O.). Five classes of loadings are given in this specification: H10, H15, H20, H15-S12, and H20-S16. These loads are illustrated in Figs. 108 and 109.

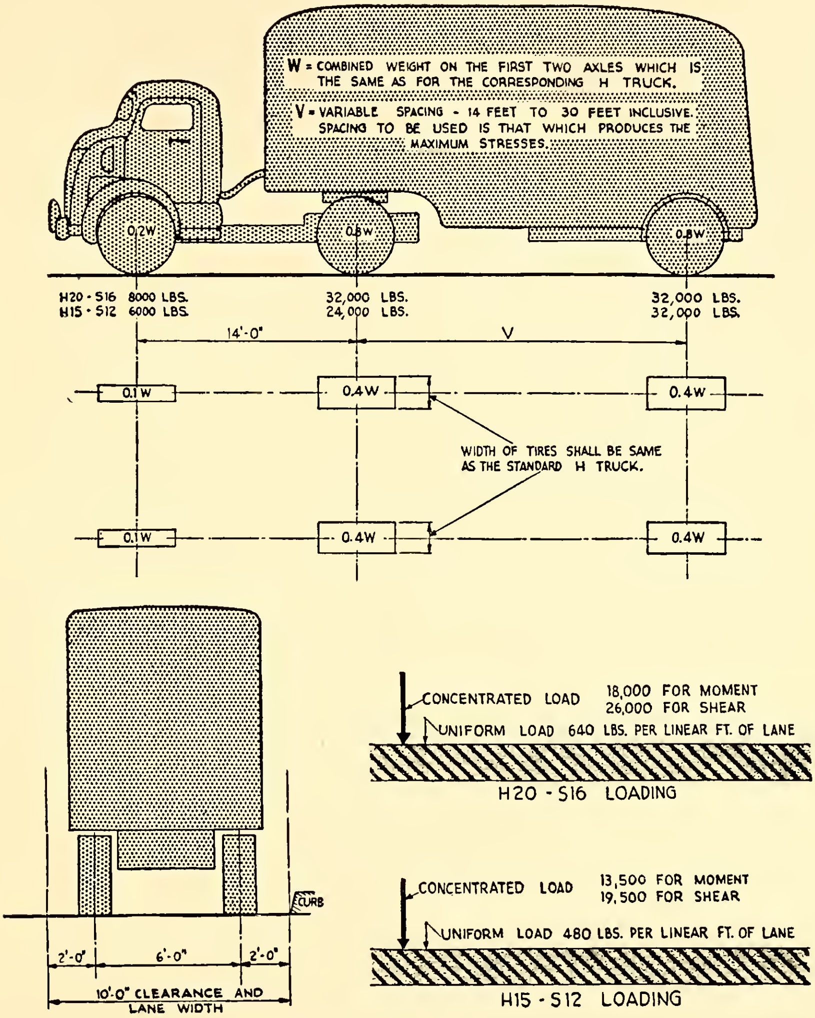

W = COMBINED WEIGHT ON THE FIRST TWO AXLES WHICH IS THE SAME AS FOR THE CORRESPONDING H TRUCK.
V = VARIABLE SPACING - 14 FEET TO 30 FEET INCLUSIVE. SPACING TO BE USED IS THAT WHICH PRODUCES THE MAXIMUM STRESSES.
0.2W
0.8W
0.8W
H20 - S16 8000 LBS.
H15 - S12 6000 LBS.
32,000 LBS.
24,000 LBS.
32,000 LBS.
32,000 LBS.
14'-0"
V
0.1W
0.4W
0.4W
WIDTH OF TIRES SHALL BE SAME AS THE STANDARD H TRUCK.
0.1W
0.4W
0.4W
CURB
2'-0"
6'-0"
2'-0"
10'-0" CLEARANCE AND LANE WIDTH
CONCENTRATED LOAD 18,000 FOR MOMENT 26,000 FOR SHEAR
UNIFORM LOAD 640 LBS. PER LINEAR FT. OF LANE
H20 - S16 LOADING
CONCENTRATED LOAD 13,500 FOR MOMENT 19,500 FOR SHEAR
UNIFORM LOAD 480 LBS. PER LINEAR FT. OF LANE
H15 - S12 LOADING

Fig. 109. Standard H-S truck and lane loadings.

The H loadings consist of a two-axle truck or the corresponding lane loading. They are designated by H followed by a number indicating the gross weight in tons of the standard truck.

The H-S loadings consist of a tractor truck with semitrailer or of the corresponding lane loading. They are designated by the letter H followed by a number indicating the gross weight in tons of the tractor truck and the letter S followed by the gross weight in tons of the single axle of the semitrailer. The variable axle spacing has been introduced in order that the spacing of axles may approximate more closely the tractor trailers now in use. The variable spacing also provides a more satisfactory loading for continuous spans in that heavy axle loads may be placed on adjoining spans to produce maximum negative moment.

The lane loadings or standard trucks should be assumed to occupy traffic lanes, each having a width of 10 ft. corresponding to the standard truck clearance width. Within the curb-to-curb width of the roadway, the traffic lanes should be assumed to occupy any position which will produce the maximum stress but which will not involve overlapping of adjacent lanes or place the center of the lane less than 5 ft. from the roadway face of the curb.

Each lane loading should consist of a uniform load per linear foot of traffic lane combined with a single concentrated load or two concentrated loads, for continuous spans, so placed on the span as to produce maximum stress. The concentrated load should be considered to be uniformly distributed across the lane on a line normal to the centerline of the lane. For the computation of moments and shears, different concentrated loads should be used as indicated in Figs. 108 and 109. The lighter concentrated loads should be used when the stresses are primarily bending stresses, and the heavier concentrated loads should be used when the stresses are primarily shearing stresses.

In computing stresses, each 10-ft. traffic lane loading or single standard truck loading should be considered as a unit occupying 10 ft. of width. Fractional lane widths or fractional trucks should not be considered.

Where maximum stresses are produced in any member by loading any number of traffic lanes simultaneously, the following percentages of the resultant live load stresses should be used in view of improbable coincident maximum loading:

	Per Cent
One or two lanes	100
Three lanes	90
Four lanes or more	75

The position and number of loaded lanes should be such that it will always produce maximum stresses.

The reduction in intensity of floor-beam loads should be determined as for main trusses or girders; the width of roadway which must be loaded to produce maximum stresses in the floor beam should be used.

157. Stringer Design. Assume a timber trestle with the distance between the centers of the bents as 20 ft., and design the stringers for an H15 loading. The roadway width is 24 ft., and the timber stringers and deck have a working stress in tension of 1450 lb. per sq. in. and weigh 50 lb. per cu. ft. The weight of the wearing surface is taken as 25 lb. per sq. ft.

Example. Assume 12 rows of 8 by 16 in. stringers.

$$\text{Spacing center to center} = \frac{23.3}{11} = 2.12 \text{ ft.}$$

Dead load:

$$\text{Stringer} = \frac{8 \times 16}{144} \times 20 \times 50 = 890 \text{ lb.}$$

$$\text{4-in. deck} = \frac{4 \times 2.12 \times 20 \times 50}{12} = 707 \text{ lb.}$$

$$\text{Wearing surface} = 2.12 \times 20 \times 25 = 1060 \text{ lb.}$$

$$\text{Total} = 2657 \text{ lb.}$$

Live load:

H15 loading, rear wheel weighs 12,000 lb.

$$W' = \frac{WS}{K'} = \frac{12{,}000 \times 2.12}{4} = 6360 \text{ lb.}$$

Moments:

$$\text{Dead load moment} = \frac{2657 \times 20 \times 12}{8} = 79{,}710 \text{ in.-lb.}$$

$$\text{Live load moment} = \frac{6360 \times 20 \times 12}{4} = 381{,}600 \text{ in.-lb.}$$

$$\text{Total} = 461{,}310 \text{ in.-lb.}$$

Fiber stresses:

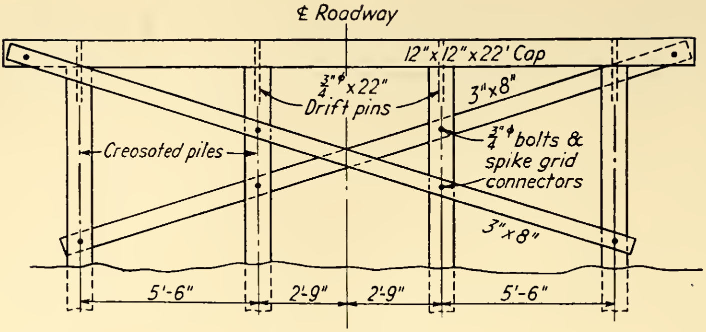
Allowable working stress = 1450 lb. per sq. in.

$$\text{Actual unit stress} = \frac{6M}{bd^2} = \frac{6 \times 461{,}310}{8 \times 16 \times 16} = 1350 \text{ lb. per sq. in.}$$

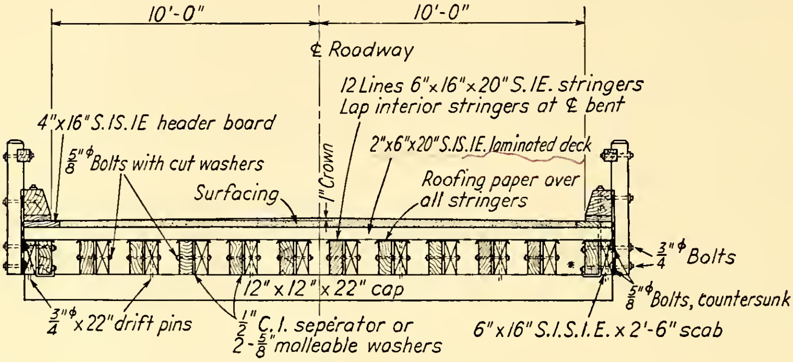
Shear: In computing the horizontal shear, place the wheel load at a point three times the depth of the beam from the support, which may be taken at the quarter point. It may be assumed that the stringer is relieved of 80 per cent of the portion of the load that is carried by the other stringers when the load is at the midspan.

Portion of load carried by other stringers when load is at midspan = 12,000 − 6360 = 5640 lb.

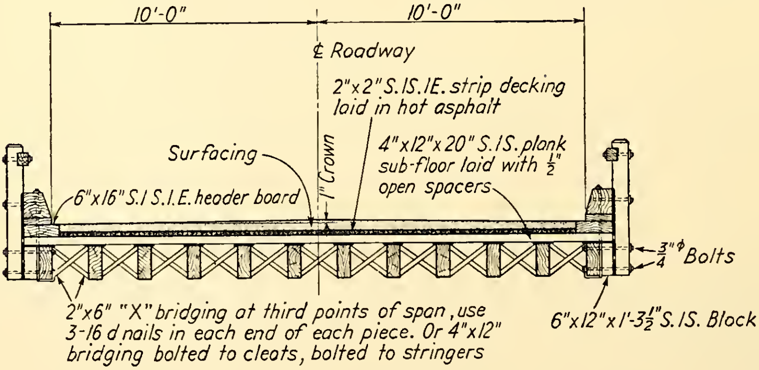
Load acting on stringer at quarter point = 12,000 − 0.8 × 5640 = 7490 lb.


℄ Roadway
12"x12"x22' Cap
3/4"φ x22" Drift pins
3"x8"
Creosoted piles
3/4"φ bolts & spike grid connectors
3"x8"
5'-6"
2'-9"
2'-9"
5'-6"

Typical Pile Bent


10'-0"
10'-0"
℄ Roadway
12 Lines 6"x16"x20" S.1E. stringers
Lap interior stringers at ℄ bent
4"x16" S.1S.1E header board
2"x6"x20" S.1S.1E. laminated deck
5/8"φ Bolts with cut washers
1" Crown
Roofing paper over all stringers
Surfacing
3/4"φ Bolts
12" x 12" x 22" cap
5/8"φ Bolts, countersunk
3/4"φ x 22" drift pins
1/2" C.I. seperator or 2-5/8" malleable washers
6"x16" S.1.S.1.E. x 2'-6" scab

Typical Section at Bent
Showing Laminated Type Deck


10'-0"
10'-0"
℄ Roadway
2"x2" S.1S.1E. strip decking laid in hot asphalt
Surfacing
1" Crown
4"x12"x20" S.1S. plank sub-floor laid with 1/2" open spacers
6"x16" S.1 S.1.E. header board
3/4"φ Bolts
2"x6" "X" bridging at third points of span, use 3-16 d nails in each end of each piece. Or 4"x12" bridging bolted to cleats, bolted to stringers
6"x12"x1'-3 1/2" S.1S. Block

Typical Section at Mid-span
Showing Strip and Plank Type Decking

FIG. 110. Timber trestle details.

In determining the shear due to dead load, the uniform load from the end of the stringer to a point equal to the height of the beam should be disregarded.

Total shear:

$$\text{Live load} = 7490 \times \tfrac{3}{4} = 5620 \text{ lb.}$$

$$\text{Dead load} = \frac{2657}{2}\left(\frac{10 - 1.3}{10}\right) = 1154 \text{ lb.}$$

$$\text{Total} = 6774 \text{ lb.}$$

Unit shear:

$$\text{Allowable} = 120 \text{ lb. per sq. in.}$$

$$\text{Actual} = \frac{3}{2}\,\frac{6774}{8 \times 16} = 79.3 \text{ lb. per sq. in.}$$

TIMBER-CONCRETE COMPOSITE DECKS

158. General. The American Wood Preservers' Association is the owner of the patent covering treated timber-concrete composite decks, but it is the policy of the Association to make this type of construction available to the public without undue restrictions.

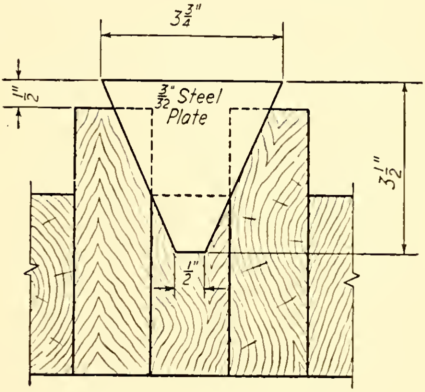


FIG. 111. Shear developers.

The treated timber-concrete composite deck is a slab consisting of a laminated timber base that is interlocked with a concrete mat. The concrete mat performs two functions: it helps support the loads, and it acts as an extremely durable wearing surface. The treated timber base is made up of 2-in. planks laid on edge with varied widths of adjacent planks. The usual method is to use a 2 by 4 in. piece with a 2 by 6 in. piece next to it, alternating the widths across the width of the roadway. Metal plates called "shear developers" are driven into the longitudinal grooves with uplift spikes driven into the raised lamination at intervals of 2 to 4 ft.

The concrete mat is usually reinforced for shrinkage and temperature stresses to prevent cracking; this reinforcement consists of ⅜-in. or ½-in. round bars placed on 9 to 12 in. centers in both directions. When the deck is continuous over a support, additional reinforcing is usually added to take care of the negative moments. Recently, however, a number of composite decks have been designed and built without any reinforcing, and they have not developed any evidence of weakness or cracks of a serious nature.

For single-span lengths all the planks should be full length, extending from support to support; but in multiple-span construction, one-third of the laminations should butt joint over the support and one-third at each quarter point. With this arrangement two-thirds of the laminations are effective at the support and at the quarter points, and a full section extends throughout the distance between these points.

159. Loads. In the construction of timber-concrete composite decks, the treated timber is assembled in place first, and the shear developers and uplift spikes are driven before the concrete mat is poured. Because the timber sub-deck carries all the dead load the first procedure in design is to determine the dead load stresses in the timber sub-deck.

Tests have shown that, in continuous spans with the laminations arranged as previously described, the positive and negative bending moments for uniform dead load in interior spans are very nearly equal and for practical purposes may be taken as 50 per cent of the simple-span dead load moment.* The positive moments in the end spans may be assumed at 60 per cent of the simple span moment.

Results of tests by the Maryland State Roads Commission make it possible to consider a critical concentrated live load as laterally distributed over 5 ft. in calculating moment. For shear, the distribution may be taken as 4 ft.

160. Design Assumptions. It is assumed that the ratio of the modulus of elasticity of concrete E_c and that of wood E_w is equal to one and that the ratio of the modulus of elasticity of steel E_s to that of southern pine or Douglas fir is 18.75. The lateral distribution for concentrated loads in calculating moment is 5 ft. and for shear is 4 ft. The distribution of the dead load bending moments in continuous spans is taken as 50 per cent of the simple span dead load moment in interior spans and as 60 per cent in end spans. The distribution of the live load bending moments is taken as 70 per cent of the simple span live load moment or positive moments in interior spans and 50 per cent for negative moments. For end spans, 85 per cent of the simple span moment is taken for positive moments and 40 per cent for negative moments.

161. Design. Assume a continuous composite deck as part of a highway bridge to carry an H15 loading. The centerlines of the bents are 20 ft. apart. Design the interior span of the composite deck using a 1-ft. width in the calculations.

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* "Timber-Concrete Composite Decks," Service Bureau, American Wood Preservers' Association, April, 1941.

Working stresses:

Timber: 1700 lb. per sq. in., tension.
Concrete: 800 lb. per sq. in., compression.
Steel: 18,000 lb. per sq. in.
Shear developers: 1750 lb. each for steel $\frac{3}{32}$ in. thick in grooves 2 in. wide or less.

Weights:

Timber: 60 lb. per cu. ft.
Concrete: 150 lb. per cu. ft.

Wood section at midspan:

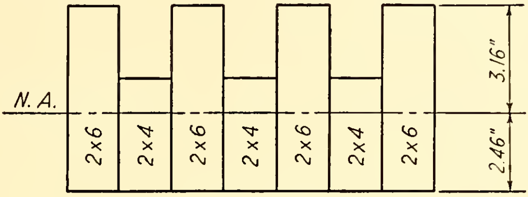


FIG. 112. Wood section at mid-span.

$$\begin{array}{lll} 6\times 5.625 = 33.75 & 33.75\times 2.812 = 95.0 & \frac{bd^3}{12} = \frac{6\times 5.625^3}{12} = 88.90 \\ 6\times 3.625 = \underline{21.75} & 21.75\times 1.812 = \underline{41.3} & \frac{6\times 3.625^3}{12} = 23.85 \\ \qquad\qquad\quad 55.50 & \qquad\qquad\quad)136.3 & \\ & \qquad\quad y = 2.46 & Ad^2 = 33.75\times 0.35^2 = 4.13 \\ & & \qquad\ \ 21.75\times 0.65^2 = \underline{9.18} \\ & & \qquad\qquad\qquad I_1 = 126.06 \end{array}$$

Wood section at support:

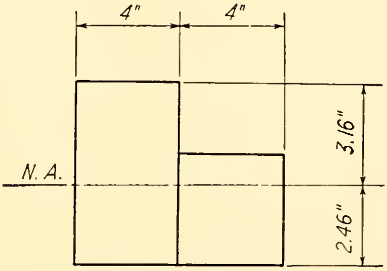


FIG. 113. Wood section at support.

$$I_2 = \tfrac{2}{3}\times 126.06 = 84.1$$

Composite section at midspan:

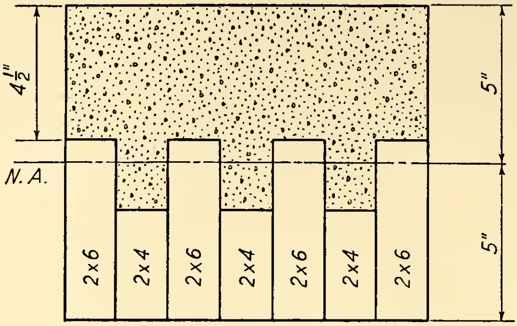


FIG. 114. Composite section at mid-span.

This section is considered homogeneous.

$$y = 5''$$

$$I_3 = \frac{bd^3}{12} = \frac{12 \times 10^3}{12} = 1000$$

Composite section at support:

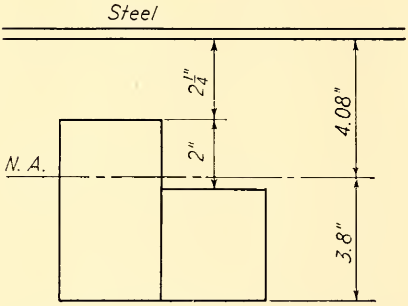


FIG. 115. Composite section at support.

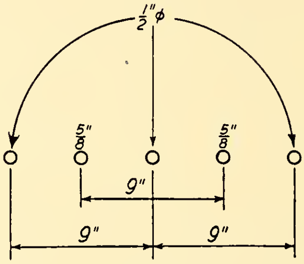


FIG. 116. Arrangement of steel.

$$\begin{array}{rcrcr}
\frac{2}{3} \times 6 \times 5.625 & = & 22.5 \times 5.06 & = & 114.0 \\
\frac{2}{3} \times 6 \times 3.625 & = & 14.5 \times 6.06 & = & 87.8 \\
 & & \underline{12.56} \times 0 & = & \underline{0} \\
 & & 49.56 & &)201.8
\end{array}$$

$$y_s = 4.08$$

$$y_w = 3.80$$

$$\frac{bd^3}{12} = \frac{4 \times 5.625^3}{12} = 59.3$$

$$\frac{4 \times 3.625^3}{12} = 15.9$$

$$\begin{array}{rcl}
\text{Steel: } \frac{5}{8} \text{ in. } \phi & = & 0.307 \times 2 = 0.614 \\
\frac{1}{2} \text{ in. } \phi & = & 0.196 \times 2 = \underline{0.392} \\
 & & 1.00
\end{array}$$

$1.00 \times \frac{12}{18} = 0.67$ sq. in. per ft. of slab width

$0.67 \times 18.75 = 12.56$ sq. in. of wood

$$\begin{aligned} Ad^2 = 22.5 \times 1^2 &= 22.5 \\ 14.5 \times 2^2 &= 58.0 \\ 12.56 \times 4.08^2 &= \underline{209.0} \\ I_4 &= 364.7 \end{aligned}$$

Dead loads and moments:

$$\text{Timber dead load} = \frac{4.625}{12} \times 60 = 23.125 \text{ lb. per sq. ft.}$$

$$\text{Concrete dead load} = \frac{5.5}{12} \times 150 = \underline{68.5} \text{ lb. per sq. ft.}$$

$$91.625 \text{ lb. per sq. ft.}$$

$$\text{Simple span dead load moment} = \frac{91.625 \times 20 \times 20 \times 12}{8} = 55{,}000 \text{ in.-lb.}$$

Distribution of moment in interior spans:

Positive moment $= 55{,}000 \times 0.5 = 27{,}500$ in.-lb.

Negative moment $= 55{,}000 \times 0.5 = 27{,}500$ in.-lb.

Live loads and moments:

Wheel load $= 12{,}000$ lb.

$$\text{Load per foot of width} = \frac{12{,}000}{5} = 2400 \text{ lb.}$$

$$\text{Simple span moment} = \frac{2400 \times 20 \times 12}{4} = 144{,}000 \text{ in.-lb.}$$

Positive moment (interior span) $= 144{,}000 \times 0.7 = 100{,}800$ in.-lb.

Negative moment (interior support) $= 144{,}000 \times 0.5 = 72{,}000$ in.-lb.

Dead load stresses:

$$\text{Tension at midspan } f_w = \frac{27{,}500 \times 2.46}{126.06} = 537 \text{ lb. per sq. in.}$$

$$\text{Compression at support } f_w = \frac{27{,}500 \times 2.46}{84.1} = 805 \text{ lb. per sq. in.}$$

Live load stresses:

Midspan:

$$\text{Tension in wood } f_w = \frac{100{,}800 \times 5}{1000} = 504 \text{ lb. per sq. in.}$$

$$\text{Compression in concrete } f_c = \frac{100{,}800 \times 5}{1000} \times 1.3 = 655 \text{ lb. per sq. in.}$$

where 30% is added for impact.

Support:

$$\text{Compression in wood } f_w = \frac{72{,}000 \times 3.8}{364.7} = 750 \text{ lb. per sq. in.}$$

$$\text{Tension in steel } f_s = \frac{72{,}000 \times 4.08}{364.7} \times 18.75 \times 1.3 = 19{,}650 \text{ lb. per sq. in.}$$

Maximum combined stresses:

Timber:

Midspan = 537 + 504 = 1041 lb. per sq. in.
Support = 805 + 750 = 1555 lb. per sq. in.

Concrete:

Midspan = 655 lb. per sq. in.

Steel:

Support = 19,650 lb. per sq. in.

Because the stress in the steel exceeds the allowable stress it will be necessary to increase the amount of steel over the support or to use a larger wood section.

Spacing of shear developers:

Lateral load distribution = 4 ft.

Critical position of load = 3 × depth of slab from support. This point is near timber joints, and net section only is considered.

Effective load per foot of width $= \frac{12{,}000}{4} = 3000$ lb.

Increase for impact $= 3000 \times 1.3 = 3900$ lb.

End reaction $= 3900 \times \frac{17}{20} = 3315$ lb.

Net section $= (10 \times 12) - (2 \times 5.625) - (2 \times 3.625) = 100.5$ sq. in.

Average unit shear $= \frac{3315}{100.5} = 33$ lb. per sq. in.

Maximum shear at neutral axis $= 33 \times 1.5 = 49.5$ lb. per sq. in.

Total horizontal shear on 1 sq. ft. $= 49.5 \times 144 = 7130$ lb.

Shear developers required per square foot of contact area $= \frac{7130}{1750} = 4.07$.

Number of grooves per foot of width $= \frac{6}{1.625} = 3.69$.

Shear developers required per linear foot in each groove $= \frac{4.07}{3.69} = 1.1$.

Spacing in each groove = 10.9 in. (use 10.5 in.). This spacing may be increased as the shear decreases along the span.

PROBLEMS

1. Design a plank floor and stringer system for a timber trestle with bents 15 ft. on centers to carry an H10 loading on an 18-ft. roadway. Use Douglas fir, 1700 f.

2. Design an A-frame truss to span 20 ft. and carry an H10 loading on a roadway 20 ft. wide. Use Douglas fir, 1700 f.

3. For a span of 50 ft. and a roadway 24 ft. wide, design and detail a truss bridge to carry an H15 loading. Use Douglas fir, 1700 f.

4. Design a continuous timber-concrete composite deck to carry an H20 loading when the bents are spaced 18 ft. apart and the roadway is 24 ft. wide. Use No. 1 dense structural southern pine.

5. Design a single span timber-concrete composite deck 22 ft. long to carry an H15 loading when the roadway is 20 ft. wide. Use No. 1 dense structural southern pine.

CHAPTER IX

GLUED LAMINATED CONSTRUCTION

162. General. Glued laminated beams and arches have been used extensively in Europe and are rapidly gaining favor in this country. They are usually built up of nominal 1- or 2-in. material with each ply glued so that the grain of all the plies is in the same direction.

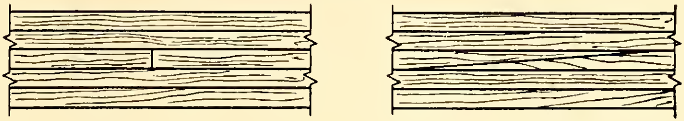
The method of gluing small boards together to form large cross sections has many advantages. Among these are (1) larger cross sections and lengths can be manufactured than those available as single pieces; (2) lumber that is not ordinarily classified as structural lumber can be used to form a cross section that will be just as strong as a solid piece; (3) the laminations can be arranged so that the highly stressed portions of a member will contain the pieces with the least number of defects and the greatest density; (4) a single cross section forming an arch rib has greater fire resistance than a truss constructed of smaller pieces designed to carry the same load; and (5) in bowstring trusses the curved top chord can be made continuous through the panel points, thus simplifying fabrication. There is one chief disadvantage, which is found in the manufacturing process. To obtain the proper bond between the laminations, the amount of pressure and the temperature must be controlled during gluing operations. This means that for best results most members are fabricated in a shop and then shipped to the job site.

163. Manufacture. The builder of glued laminated members must have the proper equipment for mixing and spreading glue, for maintaining the required temperature, and for applying the necessary pressure. Each lamination is usually spread with glue on both sides by means of mechanical spreaders. However, this operation can be performed by hand. When curved members are being built it is necessary to bend the laminations around a suitable form. Pressure is applied by means of hand-operated clamps, screw jacks, hydraulic presses, or any other suitable device.

There are several methods for joining members end to end to get the required length. These are illustrated in Fig. 117.

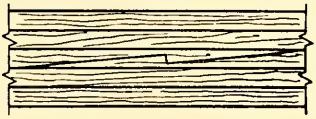
Butt joints have no strength in tension and are inadequate in resisting the bending of the laminations to the required curvature. If it is properly glued, the scarfed joint, in which the slope of the scarf is usually

1 in 12, is as strong as the wood itself. The hooked scarfed joint has the advantage of providing quick assembly because the two pieces can be quickly joined without the ends overlapping. However, this joint is not as strong in tension as the plain scarfed joint.



Butt Joint

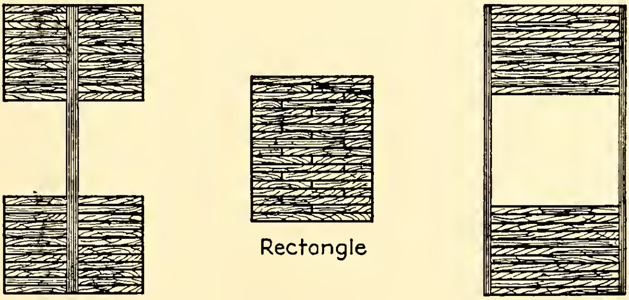
Scarf Joint



Hooked Scarf Joint

Fig. 117. Joints in laminations.

In arch construction, the arch rib is often tapered for architectural and economic reasons. There are several methods of accomplishing this taper, but the most practical is to have all the laminations running parallel to the center line of the member except the last one at each face, and it should follow the curvature of the face and be accurately fitted to the preceding laminations.



Rectangle

I-Section

Box Section

Fig. 118. Cross sections of glued laminated members.

The cross section of the member can be built up to form a rectangle, I-beam, or box-beam, but the most common type is the rectangle.

The average maximum curvature for any lamination should not be more than that determined by multiplying the thickness of the laminations by 130. Of course, some species will permit bending to a smaller radius because the stress induced during bending is different for most species.

Because the fiber stresses in any member subjected to bending moments are greatest at the outermost fiber and gradually decrease to zero at the neutral axis, it is possible to design a glued laminated member so that the innermost plies contain lumber having more defects than the face laminations. For purposes of assigning working stresses and of limiting the defects in the various laminations, the Forest Products Laboratory suggests the division of the member into three volumes.

Volumes A and C each occupy one-fifth of the cross section throughout the entire length of the member, and volume B occupies the remaining portion.

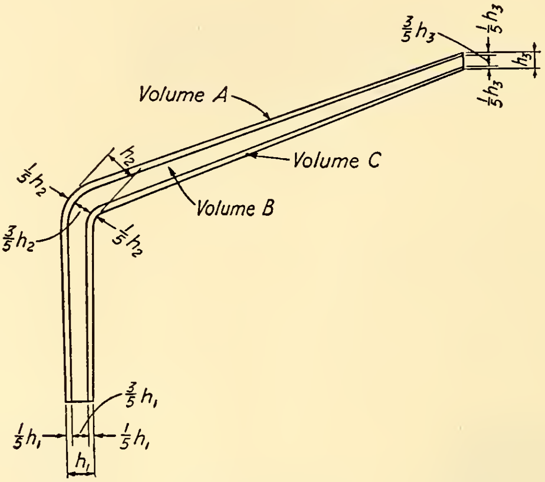


FIG. 119. Division of member into volumes.

164. Form Factor. Tests at the Forest Products Laboratory revealed that as the height of a beam increased the fiber stress at the proportional limit and the modulus of rupture decreased slightly. This makes it necessary to use a multiplier in the usual formula for internal resisting moment so that $M = FfI/c$, where F is the form factor of the section. This form factor has already been considered in the unit working stresses for the structural lumber; for rectangular sections it may be considered as unity. The formula for the resisting moment of a square wooden beam so oriented that a diagonal of the section is vertical requires a multiplier of 1.414, and the general formula is written $M = 1.414fI/c$. For a circular section the form factor is 1.18.

For I-forms and box forms, the form factor is less than unity and at the proportional limit is given by the equation

$$F = 0.58 + 0.42[K(1 - t) + t]$$

The form factor for modulus of rupture is

$$F = 0.50 + 0.50[K(1 - t) + t]$$

where t = ratio of total width of webs to the width of the section.
K = values depending on d, the ratio of the depth of the compression flange to the total depth.

The modulus of elasticity is unaffected by the shape of the cross section, and no form factor is required in computing deflections.

Table 42. Values of *K* for Different Values of *d*

d	K	d	K	d	K
0.10	0.085	0.45	0.660	0.80	0.985
0.15	0.155	0.50	0.740	0.85	0.995
0.20	0.230	0.55	0.810	0.90	0.998
0.25	0.315	0.60	0.875	0.95	1.000
0.30	0.400	0.65	0.920	1.00	1.000
0.35	0.490	0.70	0.950		
0.40	0.575	0.75	0.970		

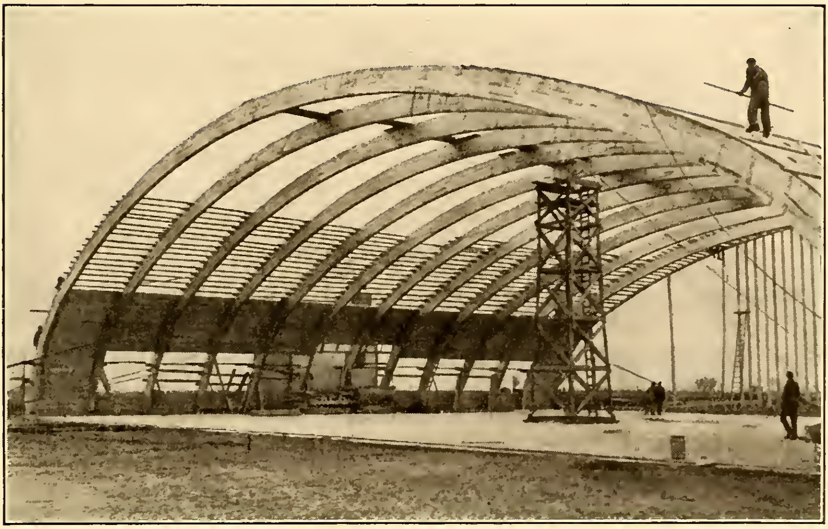


Fig. 120. 152-ft. glued laminated arches designed and fabricated by Unit Structures, Inc.

SPECIFICATIONS

The following specifications and design stresses have been prepared by the Forest Products Laboratory and appear in Bulletin 691 of the United States Department of Agriculture, under the title "The Glued Laminated Wooden Arch," by T. R. C. Wilson.

165. Moisture Content. Before being glued material must be seasoned to an average moisture content of not more than 15 per cent and not less than 10 per cent with a spread of not more than 5 per cent moisture content among pieces incorporated into a single built-up member.

166. Permissible Defects. Limitations of knots apply to both sides and limitations of slope of grain to both sides and both edges of the lamination. All laminations shall be free from shakes or splits that when viewed from the ends of the piece make an angle of less than 30° with the wide faces. Size of a knot is to be taken as the dimension of the knot between lines touching it and parallel to the edges of the lamination. Direction of grain is to be measured over a distance sufficiently great to determine the general slope, local deviations being disregarded. Material that is obviously so resinous as to be likely not to hold glue shall be rejected.

LAMINATIONS IN VOLUMES *A* AND *C*

Laminations in volumes *A* and *C* must not contain any part of the pith of the tree.

When laminations of Douglas fir, southern yellow pine, or redwood, are specified to be "close-grained" or "dense" they shall conform to the requirements for rate of growth or for rate of growth and percentage of summerwood as specified for close-grained or dense material of these species. Regardless of the grade of construction, the outer lamination on each side of the member shall be of the character specified for volumes *A* and *C* in grade I construction. Further, these laminations shall have a thickness not greater than one one-hundred-and-fiftieth of the minimum radius to which they are bent.

GRADE I CONSTRUCTION

For grade I construction, full-width or part-width laminations in volumes *A* and *C* shall be free from knots whose size exceeds one-eighth the width of the piece—maximum size 1½ in. The sum of the widths of all knots in either face in any length equal to the width of the piece shall not exceed one-eighth the width. Laminations shall be free from diagonal or spiral grain whose slope is greater than 1 in 17. Wane whose greatest width does not exceed one-half the thickness of the piece is permissible.

GRADE II CONSTRUCTION

For grade II construction, full-width or part-width laminations in volumes *A* and *C* shall be free from knots whose size exceeds one-fourth the width of the piece—maximum size 2½ in. The sum of the sizes of all knots in either face within any length equal to the width of the piece shall not exceed one-fourth the width of the piece. Laminations shall be free from diagonal or spiral grain whose slope is greater than 1 in 15. Wane whose greatest width does not exceed the finished thickness of the piece is permissible.

LAMINATIONS IN VOLUME *B*

GRADES I OR II CONSTRUCTION

Full-width or part-width laminations in volume *B* may have knotholes or sound knots whose combined widths do not exceed one-third the width of the lamination except that no defects shall be permitted that interfere with bending to the required curve without localized irregularities in the curvature or that interfere with bringing laminations into close contact. Wane whose greatest width does not exceed the finished thickness of the lamination is permissible.

LAMINATIONS IN VOLUMES *A*, *B*, AND *C*

It may be specified that all laminations shall be free from knots, knotholes, or wane that will be visible when the member is in place in the structure.

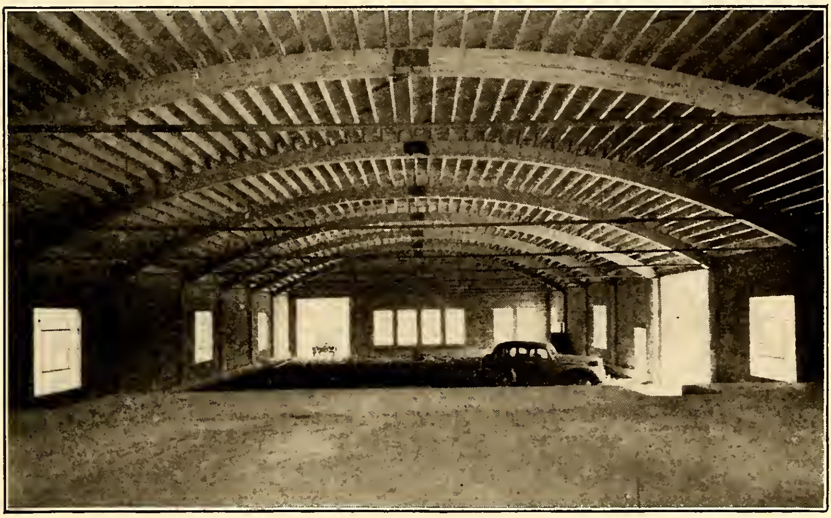


FIG. 121. Glued laminated arches spanning 58 ft. Fabricated by Rilco.

167. Preparation of Laminations. Laminations will be single piece in length or will be built up to full length of their runs by joining shorter pieces end to and, prior to the final surfacing of either side, by means of glued plain scarf joints that cross the thickness of the piece in a distance not less than 12 times the thickness for joints in volumes *A* and *C*. In volumes *A* and *C* the sloping surfaces joined to form a scarf joint must be free from knots or pitch pockets.

168. Distribution of Joints in the Width and Length of Laminations. Outer laminations shall be of one piece in width. Other laminations may be of two or more pieces provided their widths and arrangement are such that longitudinal joints in adjacent laminations are separated by at least 1½ in. Laminations or part laminations composed of two or more pieces edge glued to each other prior to the final surfacing of either side may be considered as one piece.

Scarf joints in volumes *A* and *C* shall be so arranged that at any section perpendicular to the axis of the member the sum of the widths of the joints in any group of three successive laminations shall not exceed the width of the member. Furthermore, joints showing on either edge of a member shall not be closer together, center-to-center, in adjacent laminations in volumes *A*, *B*, or *C* than 24 times the thickness of a single lamination.

Joints in the length of members making up curved portions are to be avoided in so far as possible, and laminations in volumes *A* and *B* shall not be jointed at any point where the radius of curvature to which they will be bent is less than 125 times the thickness of the lamination.

169. Arrangement or Distribution of Taper. The tapering of members that vary in depth shall be accomplished by whichever of the following methods is specified.

1. All laminations to parallel the center line of the member except the last one at each face, which shall follow the curvature of the face and shall be accurately fitted to the preceding laminations.

2. A group of outer laminations totaling at least one-fifth the depth of the member at the point of maximum depth or one-half the depth at the point of minimum depth to run parallel to each face of the member, the remaining laminations being so arranged that fitting their ends will not require cutting at a slope steeper than 1 in 12. Such fitting must be accurately done so that a good glue joint results.

3. The total taper to be approximately uniformly divided among all laminations, that is, each lamination to be tapered in the same proportion as the member itself. For example, if there are 20 laminations, each will have at each point in its length a thickness approximately one-twentieth the depth of the member at the corresponding point.

170. Machining. All surfaces to be joined by gluing shall be carefully machined so that they are true and free from humps or depressions, and at the time of gluing they shall be free of dust, dirt, or grease.

171. Glue. Only a water-resistant casein glue, or other glue known to be equal in strength and moisture resistance to best quality casein glue, is to be used. Glue must be thoroughly mixed and must be free from lumps and from excessive air bubbles.

Not less than 8 lb. of wet casein glue per 100 sq. ft. of joint area, or equivalent amounts of other types used, shall be applied. Glue shall be applied to a uniform thickness and preferably to both of the faces meeting at a joint. Glue-coated pieces shall be laid together as soon as the glue is spread.

172. Pressure. While the glue is setting all glued joints are to be subjected to a pressure of not less than 100 nor more than 200 lb. per sq. in. by means of clamps, screw jacks, presses, or other similar appliances.

Clamping must be completed within 20 min. after the first glue is spread if glue is applied to both faces meeting at a joint, or within 15 min. if glue is applied to only one of these faces. When a member is built up in installments,

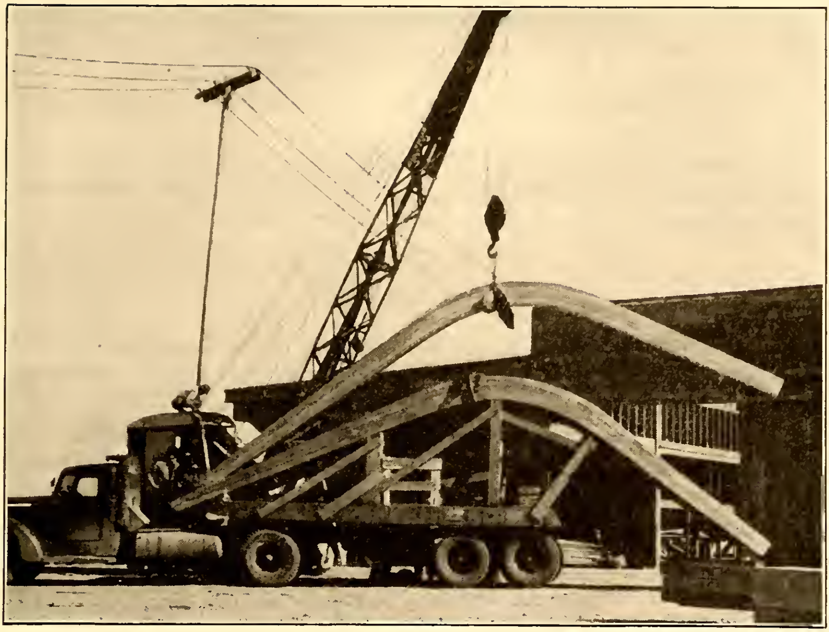


FIG. 122. Glued laminated arch segments loading at Timber Structures, Inc., for shipment to Camp White, Medford, Oregon.

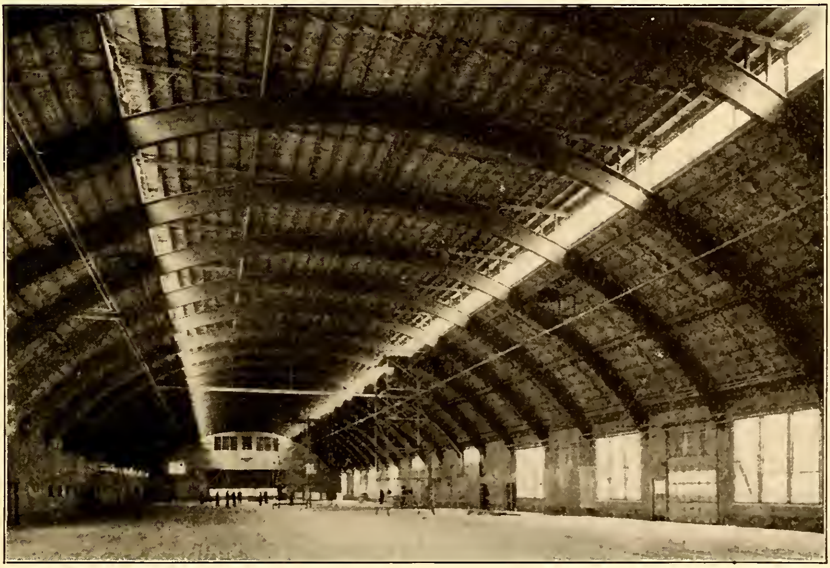


FIG. 123. Laminated arches used in navy drill halls. Fabricated by Unit Structures Inc., Peshtigo, Wisconsin.

each group or installment of laminations shall remain under pressure for at least 3 hr. before being released to add the next installment. Pressure shall be maintained at least 12 hr. after the addition of the last installment.

Special care must be taken to assure good gluing between successive installments of laminations.

173. Temperature. The room in which gluing is done shall be maintained at a temperature of 50° F. or higher. Material is to be brought approximately to the temperature of the glue room before gluing is begun.

DESIGN STRESSES

The following is suggested as the procedure for determining from Table 43 the stresses for use in the design of laminated members built in accordance with the specifications presented in the preceding section.

174. Combined Bending and Compression. The sum of the bending stress and the compressive stress parallel to the axis of the member shall not exceed the value given in Table 43, Column 2 (increased in accordance with Footnote 1 when this footnote is applicable to the material specified for use in volumes *A* and *C*), multiplied by grade of construction, curvature, and depth factors as follows:

Grade of construction factors:	
For grade I construction	1.000
For grade II construction	0.875
Curvature factor:	$1.00 - 2.00\left(\frac{t}{R}\right)^2$

where t/R is the maximum value of thickness of lamination divided by the radius to which the lamination is bent at that point in the length of the member at which the stress occurs. (No curvature factor is to be applied to stress in a straight portion of a member regardless of the curvature elsewhere.)

Depth factor: $1.07 - 0.07\sqrt{\frac{h}{2}}$

where h is the depth of the member in inches at the point under consideration.

If I- or box sections are used, further multiplication by the appropriate form factor should be made.

175. Compression Parallel to Grain. The allowable stresses in compression parallel to the grain are:

For laminations of the type specified for volumes *A* and *C* in grade I construction: The value given in Table 43, Column 4, increased for close-grained or dense material according to Footnote 1.

For laminations of the type specified for volumes *A* and *C* in grade II construction: 80 per cent of the value given in Table 43, Column 4, increased for close-grained or dense material according to Footnote 1.

Table 43. Basic Stress Values for Laminated Wooden Construction in Pounds per Square Inch [1]

Species (1)	Combined Bending and Compressive Stress in Extreme Fiber (2)	Compression Perpendicular to Grain (3)	Compression Parallel to Grain (4)	Maximum Longitudinal Shear (5)	Modulus of Elasticity (6)
Softwoods:					
Cedar, Alaska	1,759	300	1,280	144	1,200,000
Cedar, northern and southern white	1,200	210	879	112	800,000
Cedar, Port Orford	1,759	300	1,440	144	1,500,000
Cedar, western red	1,440	240	1,120	127	1,000,000
Cypress, southern	2,040	360	1,759	160	1,200,000
Douglas fir, coast region	2,400	390	1,759	144	1,600,000
Douglas fir, Rocky Mountain region	1,759	330	1,280	136	1,200,000
Fir, commercial white	1,759	360	1,120	112	1,100,000
Fir, balsam	1,440	180	1,120	112	1,000,000
Hemlock, eastern	1,759	360	1,120	112	1,100,000
Hemlock, western [2]	2,080	360	1,440	120	1,400,000
Pine, western white,[3] northern white, sugar, and ponderosa	1,440	300	1,200	136	1,000,000
Pine, red	1,759	360	1,280	136	1,200,000
Pine, southern yellow [4]	2,400	390	1,759	175	1,600,000
Pine, southern yellow, dense	2,798	455	2,063	205	1,600,000
Redwood	1,920	300	1,600	112	1,200,000
Spruce, Engelmann	1,200	210	960	112	800,000
Spruce, red, white, and Sitka	1,759	300	1,280	136	1,200,000
Tamarack	1,920	360	1,600	151	1,300,000
Hardwoods:					
Ash, commercial white	2,240	600	1,759	202	1,500,000
Ash, black	1,599	360	1,040	144	1,100,000
Beech	2,400	600	1,920	202	1,600,000
Birch, sweet and yellow	2,400	600	1,920	202	1,600,000
Chestnut	1,520	360	1,280	144	1,000,000
Elm, rock	2,400	600	1,920	202	1,300,000
Elm, American and slippery [5]	1,759	300	1,280	160	1,200,000
Gum, black and red	1,759	360	1,280	160	1,200,000
Hickory, true and pecan	3,040	720	2,400	224	1,800,000
Maple, sugar and black [6]	2,400	600	1,920	202	1,600,000
Oak, commercial red and white	2,240	600	1,600	202	1,500,000
Tupelo	1,759	360	1,250	160	1,200,000

[1] For members with laminations in volumes *A* and *C* of close-grained material of Douglas fir from the Pacific Coast region, southern yellow pine, or redwood, values in columns 2, 3, and 4 may be increased one-fifteenth. Close-grained material is defined as follows: Douglas fir from the Pacific Coast region and southern yellow pine will average on one end or the other of the piece not less than 6 or more than 20 annual growth rings per radial inch. Pieces averaging from 5 to 6 annual rings per inch should be accepted as the equivalent of close-grained if they have one-third or more summerwood. Redwood will average on one end or the other not less than 10 nor more than 25 annual growth rings per radial inch. For members with laminations in volumes *A* and *C* of dense Douglas fir or dense southern yellow pine, values in columns 2, 3, 4, and 5 may be increased one-sixth. Dense material of these species will average on one end or the other not less than 6 annual growth rings per radial inch and in addition not less than one-third summerwood.

[2] Also sold as West Coast hemlock. [3] Also sold as Idaho white pine. [4] Also sold as longleaf or shortleaf southern pine. [5] Also sold as white elm or soft elm. [6] Also sold as hard maple.

For laminations of the type specified for volume B: 75 per cent of the value given in Table 43, Column 4, without increase for close-grained or dense material.

The total thrust at any cross section shall not exceed the sum of the products of the cross-sectional areas of laminations of each type multiplied by the allowable stress in compression parallel to grain for laminations of that type.

176. Compression Perpendicular to Grain (at Bearings or Radial Compression). For allowable stresses in compression perpendicular to grain, the value taken from Table 43, Column 3, increased for close-grained or dense material according to Footnote 1, should be used.

177. Tension Perpendicular to Grain (Radial Tension). For allowable stresses in tension perpendicular to grain, the values taken from Table 43, Column 3, should be used when multiplied by one-seventh for softwood species or one-sixth for hardwood species and further multiplied by the following factors according to the type of material at the point under consideration: 0.70 for defects restricted as specified for volumes A and C in grade II construction; 0.60 for defects restricted as specified for volume B.

178. Longitudinal Shear. For allowable stresses in longitudinal shear, three-fourths of the values in Table 43, Column 5, should be used.

179. Design. Glued laminated construction is best adapted to two-hinged and three-hinged arches. Because of the ease in fabrication and erection, the majority of arches are three-hinged. The following example illustrates the method of obtaining the stresses and the dimensions of the cross section at several points in a typical three-hinged arch uniformly loaded. The writer has found by experiment that the uniform loads may be considered concentrated every 6 ft. or less without any appreciable error. This greatly reduces the work involved in obtaining the maximum stresses. However, in the following example the maximum moments, thrusts, and shears have been determined graphically by considering the dead load uniformly distributed across the span and the live load uniformly distributed on the full span and on the half span.

Table 44. Maximum Design Stresses

Section	*Moment* (*Foot-Pounds*)	*Thrust* (*Pounds*)	*Shear* (*Pounds*)
a	47,500	13,780	7,950
b	85,000	15,850	2,140
c	49,600	11,680	7,880
d	21,360	10,460	4,470
e	15,140	9,130	1,650

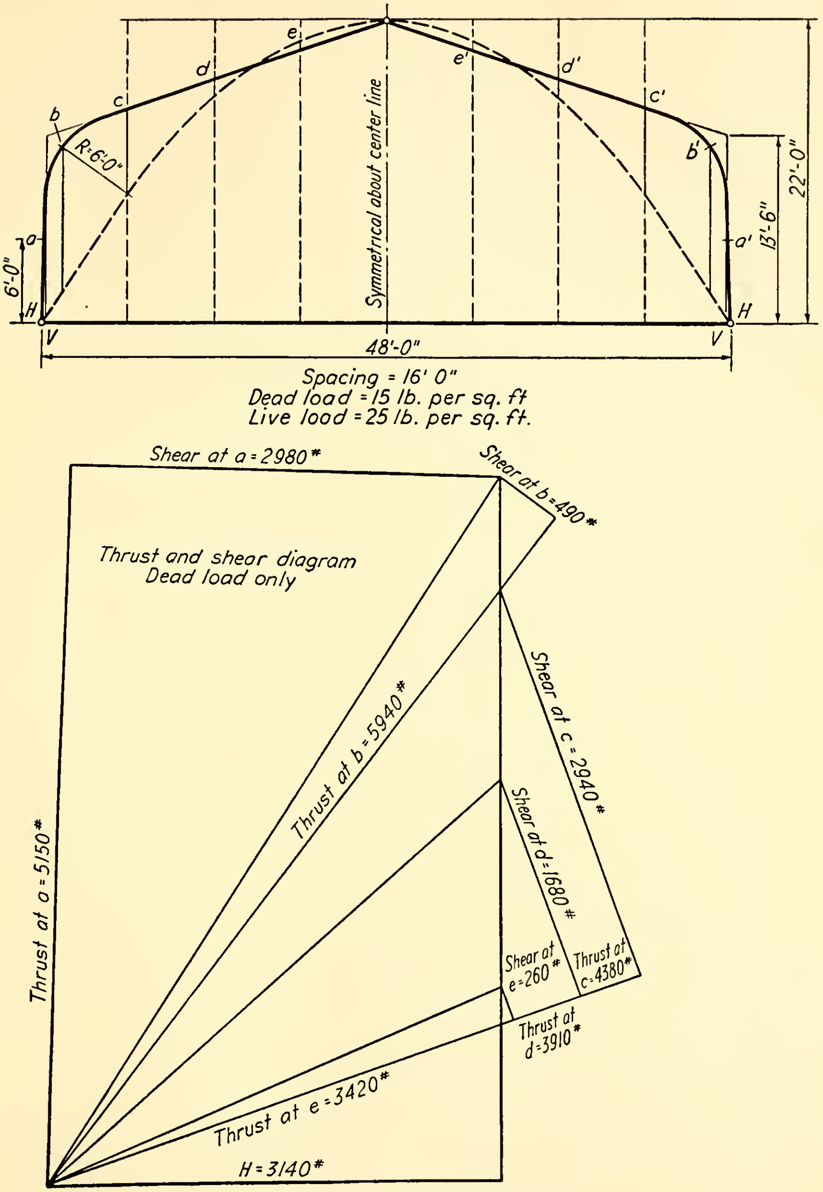

e
e'
d
d'
c
c'
b
b'
a
a'
R=6'-0"
Symmetrical about center line
6'-0"
13'-6"
22'-0"
H
H
V
V
48'-0"
Spacing = 16' 0"
Dead load = 15 lb. per sq. ft
Live load = 25 lb. per sq. ft.
Shear at a = 2980*
Shear at b = 490*
Thrust and shear diagram
Dead load only
Thrust at b = 5940*
Shear at c = 2940*
Shear at d = 1680*
Thrust at a = 5150*
Shear at e = 260*
Thrust at c = 4380*
Thrust at d = 3910*
Thrust at e = 3420*
H = 3140*

FIG. 124. Dead load stress diagram.

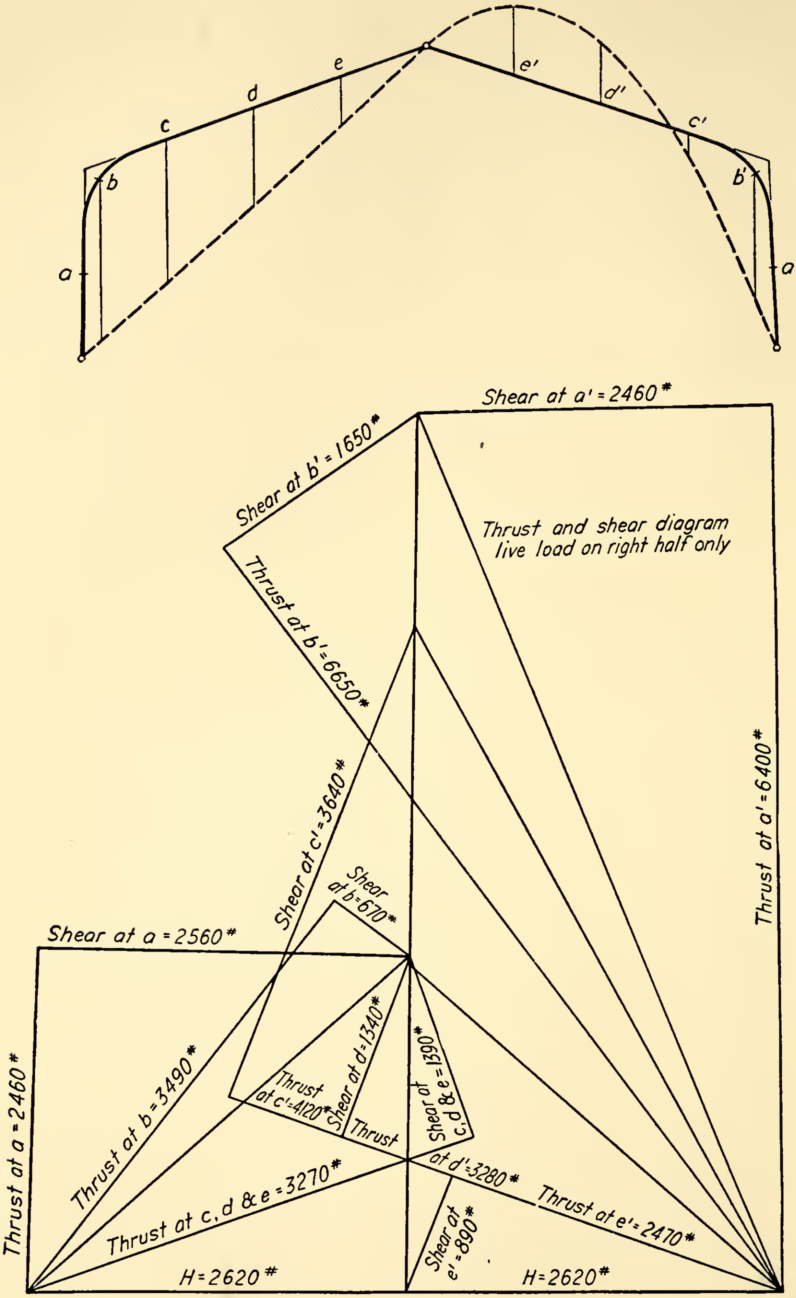

a
b
c
d
e
e'
d'
c'
b'
a'
Shear at a' = 2460*
Shear at b' = 1650*
Thrust and shear diagram
live load on right half only
Thrust at b' = 6650*
Thrust at a' = 6400*
Shear at c' = 3640*
Shear at b = 670*
Shear at a = 2560*
Thrust at a = 2460*
Thrust at b = 3490*
Thrust at c' = 4120*
Shear at d = 1340*
Shear at c, d & e = 1390*
Thrust at d' = 3280*
Thrust at e' = 2470*
Thrust at c, d & e = 3270*
Shear at e' = 890*
H = 2620*
H = 2620*

FIG. 125. Stress diagram for live load on one-half the span.

EXAMPLE. Assuming a width of 7½ in. and using close-grained 1-in. southern pine boards of a grade equal to grade I construction, consider the design of the section at point *b*.

Assume $d = 28.5$ in. $b = 7.5$ in.

Allowable $f = 2130 \times$ construction factor $\times$ curvature factor $\times$ depth factor

Construction factor $= 1.00$

$$\text{Curvature factor} = 1.00 - 2000\left(\frac{\frac{25}{32}}{72 - 14.25}\right)^2 = 0.634$$

$$\text{Depth factor} = 1.07 - 0.07\sqrt{14.25} = 0.806$$

$$\text{Allowable } f = 2130 \times 0.634 \times 0.806 = 1086 \text{ lb. per sq. in.}$$

$$\text{Actual } f = \frac{T}{A} + \frac{Mc}{I} = \frac{15{,}850}{7.5 \times 28.5} + \frac{85{,}000 \times 12 \times 6}{7.5 \times 28.5 \times 28.5} = 1077 \text{ lb. per sq. in.}$$

$$\text{Allowable shear} = 146 \times \tfrac{3}{4} = 110 \text{ lb. per sq. in.}$$

$$\text{Actual shear} = \frac{3}{2} \times \frac{2140}{7.5 \times 28.5} = 15 \text{ lb. per sq. in.}$$

It is impossible to make the final cross sections of the arch rib the exact size as determined from the maximum stresses. For architectural and practical reasons the shape of the arch rib must be modified so that a uniform taper results. This means that some sections will have very low stresses.

Table 45. Required and Final Sizes

Section	*Width* (*Inches*)	*Depth* (*Inches*) Required	Used
a	7.5	15.50	15.50
b	7.5	28.50	28.50
c	7.5	16.50	16.50
d	7.5	10.75	15.00
e	7.5	9.00	9.00

180. Radial Stress in a Curved Member. The exact solution * of the stresses in a curved beam subjected to bending moments shows that the tangential forces produce resultants in a radial direction that tend to separate the fibers when the moment is in a direction causing tension on the concave side and compression on the convex side. When the moment is in the opposite direction the radial stress is in compression. The radial stress will always be less than the fiber stress and will be a maximum near the neutral axis.

The design stresses for the fiber stresses due to bending moments in a curved laminated member have been derived on the assumption that linear distribution of bending stress exists. Of course, this assumption is incorrect, a fact recognized by Mr. Wilson,* but, because the design stresses are based on this assumption, it is believed that very little error will result by using a linear distribution of bending stress in determining the maximum fiber stress due to external loads. This value, which we shall call f_x, is less than the true stress.

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* *Theory of Elasticity*, by S. Timoshenko, p. 58, 1934.

In determining the radial stress it is necessary to know the location in the member where the maximum radial stress occurs and then calculate the radial stress by the formula obtained from the exact solution of the stresses existing in a curved member. This method is rather laborious, and for convenience Chart 36 has been prepared.

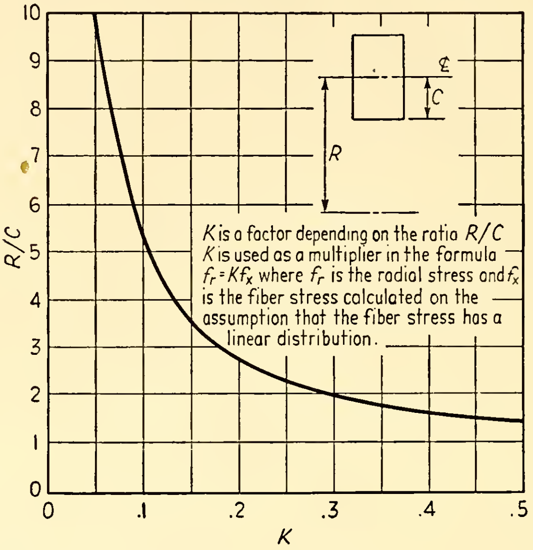

CHART 36. Values of K for determining the radial stress in a curved member subjected to bending moments.

The factor K is a combination of two constants multiplied together. The fact that the fiber stress obtained when a linear distribution of stress is assumed to be less than the actual maximum fiber stress makes it necessary to multiply by a constant to determine the true stress. This constant depends upon the ratio of the radius of curvature to the depth of the member. The radial stress is some percentage of the true stress and also depends upon the ratio of the radius of curvature and the depth of the member. K, then, combines these two constants and becomes a value to be used as a multiplier of f_x, which is the fiber stress based on a linear distribution of bending stress. The radial stress f_r is equal to Kf_x.

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* "The Glued Laminated Wooden Arch," by T. R. C. Wilson, U. S. Dept. Agr., *Bull.* 691, 1939.

The radial stress at point b in the previous example is in compression and acts perpendicular to the grain of the member. It may be calculated as follows:

$$R = 72 \text{ in.} \qquad c = 14.25 \text{ in.}$$

$$\frac{R}{c} = \frac{72}{14.25} = 5.05$$

For this value, K from Chart 11 is 0.106

$$f_r = Kf_x = 0.106 \times 1003 = 106.3 \text{ lb. per sq. in.}$$

The allowable compression perpendicular to the grain = 346 lb. per sq. in.

181. Typical Details. The following figures illustrate the methods used in joining laminated members at the crown and at the walls or abutments.

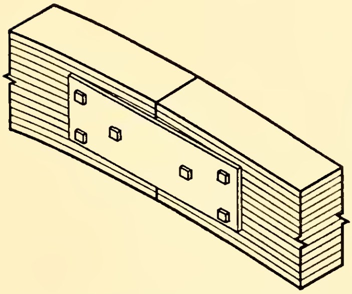

FIG. 126. Typical crown joint.

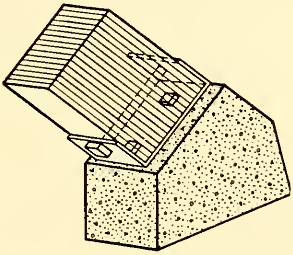

FIG. 127. Method of attaching member to abutment.

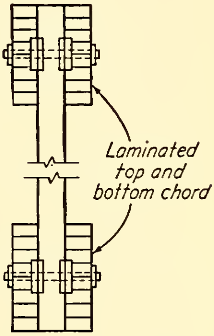

FIG. 128. Cross section of a bowstring truss.

NATIONAL DESIGN SPECIFICATIONS

182. General. Since the publication of Bulletin 691, "The Glued Laminated Wooden Arch," by T. R. C. Wilson, the National Lumber Manufacturers Association has recommended specifications for the design and construction of glued laminated members. These specifications are based on the use of members built up of laminations of seasoned stress grades of joist and plank (Table 8 in Chapter III) or of seasoned boards graded on joist and plank grading rules. This eliminates the necessity of writing special grades to adhere to the specifications set up in the previous articles.

183. Lumber. At the time of gluing, the moisture content of all laminations in a member shall be uniform and as close as is practical to the moisture content at which the member will be in service.

All surfaces to be glued shall be surfaced flat to as smooth a finish as possible (approaching a machine-polished surface but not sanded) after they have been seasoned to the appropriate moisture content.

Surfaces to be glued shall be flat, clean, and free from oil and dust. Material shall be free from warp which will prevent surfaces of adjacent laminations from being brought into intimate contact which will assure a proper thickness and uniform spread of glue. In curved portions of a member, the material shall be free from warp and from knots or knotholes which will prevent bending of laminations to uniform curvature.

Before it is assembled, lumber shall be sorted so that the material in all laminations of an individual member shall be either flat grain (angle of annual ring with wide face of less than 45°) or edge grain (angle of annual ring with wide face of 45° or more).

184. Glues and Gluing. Glue shall be of a quality which will develop the full strength of the wood used.

Only glue which will provide adequate strength under conditions of service and for the expected service life of the structure shall be used.

Good glue bond shall be made over the entire surface to be glued.

185. Joints in Laminations. The following specifications should be followed in making joints in laminations.

1. The slope of scarfed joints where laminations are fully stressed should not be steeper than that permitted for slope of grain in the grade of lumber used, except that a slope of scarf which is flatter than 1 to 12 should not be required for straight members or curved members where the radius of curvature exceeds 250 times the thickness of lamination.
2. Any end-to-end joints in exterior laminations must be scarfed.
3. Scarfed joints should be glued, the glue set, and scarfed laminations in the outer fifth dressed to a uniform thickness before they are finally assembled into the member.
4. In adjacent laminations and in corresponding laminations located symmetrically with respect to the tension and compression faces of a member, the scarfed end joints of the laminations should be staggered longitudinally by at least 24 times the thickness of a lamination, and butt end joints should be staggered by at least 40 times (measured center to center of joints).
5. For wide members a lamination may consist of two or more pieces placed side by side to form the required width, but all longitudinal joints in adjacent laminations should be staggered at least 1½ in. laterally. In width, the top and bottom face laminations should be of one piece or of pieces glued together edgewise with the excess glue removed from the surfaces prior to final assembly into the member.
6. For laminations consisting of two or more pieces in width, the sum of the widths (parallel to the width of lamination) of scarfed or butted end joints in

any three adjacent laminations, at any cross section, should not exceed the overall width of one lamination.

7. When stepped-scarf end joints are used, the slope of scarf in fully stressed laminations should not be steeper than for other scarfed joints, and any reduction in the effective cross section of the lamination, caused by the step, should be taken into consideration in the design.

8. A laminated member with scarfed joints properly made and placed should be designed as a solid member.

9. When butt end joints are used, they should not be assumed to transfer load from one side of the end joint to the other side within the laminations. The load should be assumed to be transferred to and carried by adjacent laminations.

10. When laminations are glued over tapered pieces, the taper should not exceed a slope of 1 to 12.

11. The minimum practicable number of end joints of laminations should occur at any given cross section of a member. At any highly stressed cross section, there should not be more than one end joint in any three successive laminations.

186. Curved Laminations. Members built up to desired curvatures should be limited in accordance with the following specifications.

1. No defects should be permitted which interfere with bending to the required curvature without localized irregularities in the curvature or which interfere with bringing the laminations into intimate contact.

2. For portions of members in which the laminations are curved to a radius of less than 150 times the thickness of lamination, the slope of grain in the laminations within one-tenth of the depth of the member from the outer face of each outer lamination should not exceed 1 to 15.

3. There should be no end joints in laminations where the radius of curvature is equal to or less than 150 times the thickness of lamination. End joints in laminations where the radius of curvature is between 150 and 250 times the thickness of lamination should be scarfed.

187. Working Stresses. For a member built of laminations consisting of seasoned stress grades of joist and plank or seasoned boards graded on joist and plank grading rules, the following working stresses apply.

Flexure. The allowable unit stresses in flexure for straight members or straight portions of curved members shall be equal to the allowable unit stresses in extreme fiber in bending, given for the corresponding joist and plank grades in Table 8, p. 34.

Fiber Stress for Curved Members. For the curved portion of laminated members, the allowable fiber stress in bending shall be modified by multiplication by the following factor.

$$1 - 2000\left(\frac{t}{R}\right)^2$$

where t = thickness of lamination in inches.
R = radius of curvature in inches.

Compression Parallel to Grain. The allowable unit stresses for compression parallel to grain shall be the value in Table 8 for the grade and species increased by 50 per cent, except that these stresses shall not exceed 90 per cent of the values in column A of Table 46.

Table 46. Compression Parallel to Grain and Shear Stresses

Species	A Pounds per Square Inch	B Pounds per Square Inch
Ash, white	2200	200
Beech	2400	200
Birch	2400	200
Chestnut	1600	145
Cypress, southern	2200	160
Cypress, tidewater	2200	160
Douglas fir, all regions, dense	2575	170
Douglas fir, coast region, close-grained	2350	145
Douglas fir, coast region	2200	145
Douglas fir, inland region, close-grained	2225	145
Douglas fir, inland region	2075	145
Elm, rock	2400	200
Elm, soft	1600	160
Gum, black and red	1600	160
Hemlock, eastern	1400	110
Hemlock, western	1800	120
Hickory	3000	225
Larch, dense	2575	185
Larch, close-grained	2350	160
Larch	2200	160
Maple, hard	2400	200
Oak, red and white	2000	200
Pecan	3000	225
Pine, Norway	1600	135
Pine, southern longleaf	2575	205
Pine, southern shortleaf, dense	2575	205
Pine, southern shortleaf	2200	175
Poplar, yellow	1600	130
Redwood, close-grained	2125	110
Redwood	2000	110
Spruce, eastern	1600	135
Tupelo	1600	160

Shear. The allowable unit shearing stress given in column B of Table 46 applies to all grades of a given species and density.

Combined Stresses. Laminated members subjected to both flexure and axial compression may be designed by the exact formulas given in Chapter VI or should be so proportioned that

$$\frac{\frac{P}{A}}{C} + \frac{\frac{M}{S}}{f} \text{ does not exceed } 1$$

where P = total axial load in pounds.
A = area of cross section in square inches.
M = total bending moment in inch-pounds.
S = section modulus in inches^3.
C = allowable unit stress in compression parallel to the grain that would be permitted if axial compression only existed in pounds per square inch.
f = allowable unit stress in bending if flexure only existed in pounds per square inch.

Tension Perpendicular to Grain. The allowable unit stress in tension perpendicular to the grain shall not exceed 2½ per cent of the allowable bending stress for softwoods nor 4 per cent for hardwoods.

Division of Member into Volumes. In a laminated member stressed principally in bending, the middle three-fifths of the cross section may contain material that is one grade lower than that in the upper and lower fifths of the depth, and the member may still be designed on the basis of the allowable working stresses for the material in the outer fifths of the cross section.

If the laminations in the outer tenth of a member in bending are so selected that the knots do not exceed one-half the size of the knot permitted in the joist and plank grade used elsewhere, the allowable unit stress in bending for the next higher grade in Table 8 may be used in design.

PROBLEMS

1. Design a glued laminated beam to span 30 ft. and to carry a uniformly distributed load of 1000 lb. per ft. Use western hemlock, and assume the thickness of each lamination as 1 in. (nominal).

2. A glued laminated beam simply supported between two points 40 ft. apart carries a concentrated load of 1000 lb. at the center and a uniform load of 800 lb. per ft. The beam is made of 1 by 12 in. Douglas fir boards. How many boards are required?

3. A laminated member is built up to a radius of 8 ft. The laminations are 1 by 8 in. dense southern pine and the member is subjected to a bending moment of 95,000 ft.-lb., a thrust of 17,000 lb., and a shear of 3000 lb. How many laminations are required?

4. Design a three-hinged glued laminated arch to span 70 ft. with a rise of 9 ft. 6 in. and a radius of 73 ft. to support a total load of 40 lb. per sq. ft. of horizontal surface. The arches are spaced center-to-center every 16 ft. Use 1-in. Douglas fir.

5. Design a three-hinged glued laminated arch to span 60 ft. with a total rise of 24 ft. The side legs are 14 ft. high. The arch is to carry a wind load of 20 lb. per sq. ft. of vertical surface, snow load of 20 lb. per sq. ft., and dead load of 15 lb. per sq. ft. of horizontal surface. The arches are spaced 12 ft. apart. Use 1-in. southern pine.

6. Using a top chord of two members built up of 2-in. Douglas fir and split-ring connectors, design a bowstring truss to carry a total load of 40 lb. per sq. ft. of horizontal surface. The trusses are spaced 16 ft. apart and span 80 ft.

CHAPTER X

PLYWOOD

188. General. Plywood in its simplest form consists of three layers of thin veneer glued together with the grain of the middle ply perpendicular to the grain of the face plies. The advantages in laying up veneers with the grain of adjacent plies perpendicular to each other are apparent when we consider that wood has its greatest strength along the grain. The plywood panel tends to equalize the strength in both directions and has a very high resistance to splitting by nails, screws, or other fastenings.

Because wood shrinks more across the grain and very little along the grain, plywood panels are made up of an uneven number of veneers. If this were not done the panels would warp considerably. The two outside plies are called faces, or face and back, and the center ply is called the core. Any intervening plies are cross bands or inner bands.

Plywood can be made from a variety of species that can be combined to make the best product for the use intended. Moreover, the direction of the grain in the various plies can be made at any angle to fit the needs of design, and almost any curvature can be obtained. At present plywood is used in the manufacture of furniture, boxes and crates, doors, boats, and many industrial products. In the structural field it is used primarily for floors, partitions, and sheathing, and for forms for concrete, gusset plates in large timber structures and for webs for plywood girders and arches. In the aircraft industry the use of plywood seems to be unlimited. It is being used to advantage for wing and fuselage coverings, leading edges, gusset plates, ribs, webs in box beams, nose and tail pieces, floors, walls, and numerous other parts.

MANUFACTURE

189. Veneers. In manufacturing plywood it is first necessary to cut the veneers to the desired thickness. Most veneers are rotary cut by placing a steamed log on a rotating lathe. As the lathe revolves, knives peel off a continuous layer of wood between $\frac{1}{40}$ in. and $\frac{1}{4}$ in. in thickness. These are customary thicknesses, but thinner or thicker veneers can be cut, depending upon the species of wood. Veneers are also sliced off the log when a figured pattern or edge-grained material is desired.

After the veneers are cut they are stacked and dried to a moisture content of approximately 10 per cent. Then they are flattened, and the edges are trimmed or clipped so that the smaller pieces can be taped together before assembling them into plywood.

The weight of veneers depends upon the species, thickness, and moisture content. Table 47 may be used as a guide in calculating the weight of a plywood panel.

190. Glues. The oldest form of glue is that made from the hides and bones of animals. This type is not used to any extent in the manufacture of plywood because the better grades are expensive and the glue is not water resistant.

Vegetable glues are made from starch either with or without certain chemicals added. The mixtures are relatively cheap and can be used cold; however, they are not used very often in making plywood because they are not water resistant and often cause discoloring of certain species.

Casein glues, which are made from the curd of milk mixed with lime or sodium salt, have a high degree of water resistance. They are mixed and applied when cold and are particularly useful in the construction of glued laminated members. Practically all members of this type have been made with some form of casein glue. These glues are also used for certain forms of plywood.

The blood albumin glues made by combining the blood of animals with chemicals, such as lime, caustic soda, and sodium silicate, were the first glues requiring heat to make them set. These glues are fairly waterproof but have the disadvantages of staining certain veneers, of emitting an objectionable odor, and of producing joints of a low dry strength.

The recent development of the phenolic resin glues has revolutionized the plywood industry. Two types of these glues are manufactured, the phenol formaldehyde and the urea formaldehyde. Both are furnished either as a liquid or as dry powder. The phenol resins are more durable than the urea resins. Both are, however, extremely durable under severe conditions of exposure and also possess a high resistance to mold or fungus growth. They are boilproof and can be soaked in water indefinitely and will almost always outlast unprotected wood.

The phenol resins have the added advantage of being made as a continuous film. This greatly reduces the work involved in laying up veneers for the manufacture of plywood. Moreover, extremely thin veneers are difficult to spread with a liquid adhesive, but the thin film of phenol resin can be readily placed between them.

191. Heat and Pressure. The most common form of applying heat and pressure simultaneously in the manufacture of plywood is by means of a hot-plate press which consists of a number of platens through which

Table 47. Weights of Veneers

[Ounces per Square Foot, Based on Oven-Dry]

Species	Specific Gravity Oven-Dry Weight and Volume at 12% Moisture Content	Veneer Thickness, in Inches																	
		1/100	1/80	1/64	1/60	1/55	1/48	1/40	1/32	1/28	1/24	1/20	1/16	1/12	1/10	1/8	1/6	3/16	1/4
Basswood	0.37	0.31	0.38	0.48	0.51	0.56	0.64	0.77	0.96	1.10	1.28	1.54	1.92	2.57	3.08	3.85	5.13	5.77	7.70
Beech	.62	.52	.64	.81	.86	.94	1.07	1.29	1.61	1.84	2.15	2.58	3.22	4.30	5.16	6.45	8.60	9.67	12.90
Birch, sweet	.65	.54	.68	.84	.90	.98	1.13	1.35	1.69	1.93	2.25	2.70	3.38	4.51	5.41	6.76	9.02	10.14	13.52
Birch, yellow	.62	.52	.64	.81	.86	.94	1.07	1.29	1.61	1.84	2.15	2.58	3.22	4.30	5.16	6.45	8.60	9.67	12.90
Cedar, Port Orford	.42	.35	.44	.55	.58	.64	0.73	0.87	1.09	1.25	1.46	1.75	2.18	2.91	3.49	4.37	5.82	6.55	8.74
Douglas fir (coast region)	.48	.40	.50	.62	.67	.73	.83	1.00	1.25	1.43	1.66	2.00	2.50	3.33	3.99	4.99	6.66	7.49	9.98
Gum, red	.49	.41	.51	.64	.68	.74	.85	1.02	1.27	1.46	1.70	2.04	2.55	3.40	4.08	5.10	6.80	7.64	10.19
Gum, tupelo	.50	.42	.52	.65	.69	.76	.87	1.04	1.30	1.49	1.73	2.08	2.60	3.47	4.16	5.20	6.94	7.80	10.40
Magnolia, southern	.50	.42	.52	.65	.69	.76	.87	1.04	1.30	1.49	1.73	2.08	2.60	3.47	4.16	5.20	6.94	7.80	10.40
Mahogany, African	.45	.37	.47	.58	.62	.68	.78	0.94	1.17	1.34	1.56	1.87	2.34	3.12	3.74	4.68	6.24	7.02	9.36
Mahogany, Central American	.46	.38	.48	.60	.64	.70	.80	.96	1.20	1.37	1.60	1.91	2.39	3.19	3.83	4.78	6.38	7.18	9.57
Maple, soft	.47	.39	.49	.61	.65	.71	.81	.98	1.22	1.40	1.63	1.96	2.44	3.26	3.91	4.89	6.52	7.33	9.78
Maple, hard	.63	.52	.66	.82	.87	.95	1.09	1.31	1.64	1.87	2.18	2.62	3.28	4.37	5.24	6.55	8.74	9.83	13.10
Spruce, red, white, Sitka	.39	.32	.41	.51	.54	.59	0.68	0.81	1.01	1.16	1.35	1.62	2.03	2.70	3.24	4.06	5.41	6.08	8.11
Walnut, black	.55	.46	.57	.72	.76	.83	.95	1.14	1.43	1.63	1.91	2.29	2.86	3.81	4.58	5.72	7.63	8.58	11.44
Yellow poplar	.40	.33	.42	.52	.55	.61	.69	0.83	1.04	1.19	1.39	1.66	2.08	2.77	3.33	4.16	5.55	6.24	8.32

NOTES: The weights for any given moisture content can be found by multiplying the weights given above by 1 plus the particular moisture content.

In calculating the weight of plywood for any combination of veneers, the weight of each glue line must be added. The approximate weight of phenolic resin glue per square foot of single glue line is 0.2 ounce.

Example: To find the weight of a 3-ply panel consisting of 2 face plys of 1/32-in. basswood and a core of 1/20-in. poplar glued with phenolic resin film at a moisture content of 12 per cent proceed as follows:

$$\text{Weight per square foot} = [2 \times 0.96 + 1 \times 1.66]\,1.12 + 2 \times 0.2 = 4.41 \text{ ounces}$$

Table 48. Comparison of the Properties of Various Glues[1]

[Glues for use in aircraft, Bulletin No. 1337, Forest Products Laboratory]

Property or Characteristic	Casein Glue	Blood Albumin Glue	Synthetic Resin Glue
Strength (dry)[2]	Very high to high	High to low	Very high to high
Strength (wet after soaking in water 48 hr.).[3]	About 25 to 50 per cent of dry strength—varies with glue.	About 50 to nearly 100 per cent of dry strength.	Very high; nearly 100 per cent of dry strength.
Durability in 100 per cent relative humidity or prolonged soaking in water.	Deteriorates eventually—rate varies with glue.	Deteriorates slowly but usually completely in time.	Very high if resin is unadulterated.
Rate of setting	Rapid	Very fast with heat	Very fast with heat
Working life	Few hours to a day	Few to many hours	Few to several hours for liquid forms; several weeks for films.
Consistency of mixed glue	Medium to thick; little change with temperature.	Thin to thick; little change with temperature.	Medium for liquid forms
Temperature requirements	Unimportant	Heat required to set most glues	Heat required for most glues
Mixing and application	Mixed cold with water; applied cold by hand or mechanical spreaders.	Usually mixed cold with water; applied cold by hand or mechanical spreaders.	Often applied as received or after addition of "catalyst"; liquid forms best applied by rubber-covered rolls.
Tendency to foam	Slight if not mixed too rapidly.	Slight to pronounced	Slight
Tendency to stain wood	Pronounced with certain woods	None, except dark glue may show through thin veneers.	None, although glue may penetrate through thin or porous veneers.
Dulling effect on tools	Moderate to pronounced	Slight	Moderate
Spreading capacity,[4]			
Extremes reported[5]	35 to 80 per cent	30 to 100 per cent	30 to 100 per cent
Common range[5]	40 to 60 per cent	. . . .	35 to 50 per cent

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[1] Grades and quality only of glues that pass aircraft specifications.

[2] Based chiefly on joint strength tests.

[3] Based on plywood strength tests.

[4] Expressed in square feet of single glue line per pound of dry glue for veneer work.

[5] Based on reports from manufacturers of various commercial products.

steam is circulated. These presses usually have openings 50 by 100 in. which produce finished flat panels 4 by 8 ft. The pressures used range from 150 to 300 lb. per sq. in., and the temperatures range from 230° to 320° F., depending upon the species.

Pressures can also be applied by using flexible bags in conjunction with molding forms or dies. These bags can be used on the outside or inside of a mold to form such curved parts as leading edges for airplane wings or boat hulls. The medium of pressure is usually in the form of hot water within the bag with additional heat circulating in the wooden or steel mold. This process has particular advantages for the manufacture of curved plywood because the veneers can be bent before they are glued, thus eliminating the stresses that are ordinarily set up in plywood that is bent after manufacture.

A recent development in the manufacture of plywood is known as a high frequency method. In this process the heat required to produce the bond between the veneers is created by an electric current oscillating at a very high frequency. Since the capacity of a condenser is measured by the amount of electricity that it is capable of storing and since this amount varies with the kind of dielectric between the plates, it is readily conceived that wood will absorb more energy in the form of heat when it is wet than when it is dry because dry wood has a dielectric constant of 3.6 and water has a dielectric constant of 81. This also means that the greatest heating effect will be in the wettest portion of the wood, and in making plywood the glue line is the wettest portion. By the high frequency method the wet glue can be made hotter than the wood, and danger to the wood is eliminated because the overdrying of veneers during the short time required to cure the resin bond is improbable. In the manufacturing operation the electrodes are placed on opposite sides of the plywood assembly, and the pressure is provided mechanically. Up to the present this manufacturing process has not been thoroughly investigated, but indications point to an increase in the speed of manufacturing thick panels and in adaptions for making molded parts.

192. High Density Plywood. Hot-pressed thermosetting resin adhesives for the manufacturing of waterproof plywood require pressures that seldom exceed 200 lb. per sq. in., and the total compression of the wood or reduction in thickness is rarely more than 10 per cent. Recently, however, the possibility of increasing the pressures in the manufacture of resin-bonded plywood has opened up an entirely new field. Tests have been made on plywood subjected to pressures of 500 to 1500 lb. per sq. in., and today several manufacturers are making this high density or compressed plywood.

By an increase of the pressure applied in the hot-plate process, a reduction in thickness results which means that there is more wood fiber per square inch of cross section, and the compressed product will be stronger per square inch than ordinary plywood. The strength-weight ratio of compressed plywood makes it an ideal material for airplane construction. Its properties can be used to advantage in the design of reinforcing plates for the attachment of spars to the fuselage, gusset plates for ribs, hinges, floors, instrument panels, and seats. There are many other industrial uses, such as ribs and stems for boats, gun stocks, implement handles, gears, and dies.

It is also possible to manufacture a variable density plywood; that is, one that has extra laminations at one end. In this type, one end will be stronger than the other, but the entire panel will have the same thickness. The finished product has distinct advantages in propeller construction because the high density section can be attached at the hub where strength is needed, and the low density section toward the tip of the propeller reduces centrifugal forces.

Compressed plywood is manufactured in the same manner as ordinary plywood, except that higher pressures are used. It is also possible to make an impregnated plywood. In this type, the veneers are soaked in a phenolic resin solution until the desired impregnation is obtained. Of course, the time required for impregnation can be reduced by mechanical means. After impregnation the surplus resin is evaporated, and the veneers are assembled into plywood under heat and pressure. Lower pressures can be used with impregnated veneers to obtain the same amount of compression as that developed in resin film plywood. However, the manufacture of compressed plywood with liquid resin impregnates is more complex and requires more time and care.

DOUGLAS FIR PLYWOOD

193. Sizes. Douglas fir is the species used for manufacturing most plywood that is intended for sheathing, flooring, forms for concrete, gusset plates in timber trusses and arches, and webs in plywood girders. It is available in standard sizes up to 48 in. wide and 96 in. long and in various thicknesses of $\frac{3}{16}$ to $1\frac{3}{16}$ in. and in scarf-jointed smaller panels of practically any size. There are also a few manufacturers capable of producing panels 8 ft. wide and 16 ft. long.

194. Grades. Douglas fir plywood is divided into two general classes which depend upon the use for which the panel is intended. One type is made with a phenolic resin adhesive and is meant for exterior use because it is waterproof. The other is made with an ordinary moisture-resistant

Table 49. Veneer Thicknesses for Douglas Fir Plywood Panels

[R, rough; S, sanded]

Plywood Thickness (net)	No. of Plies	Veneer Thickness (Nominal, in Inches)		
		Faces	Centers	Cross Band
$\frac{1}{8}$″—R	3	$\frac{1}{24}$	$\frac{1}{24}$	
$\frac{1}{8}$″—S	3	$\frac{1}{16}$	$\frac{1}{16}$	
$\frac{3}{16}$″—R	3	$\frac{1}{16}$	$\frac{1}{16}$	
$\frac{3}{16}$″—S	3	$\frac{1}{12}$	$\frac{1}{12}$	
$\frac{1}{4}$″—R	3	$\frac{1}{12}$	$\frac{1}{12}$	
$\frac{1}{4}$″—S	3	$\frac{1}{9}$	$\frac{1}{9}$	
$\frac{5}{16}$″—R	3	$\frac{1}{10}+$	$\frac{1}{10}+$	
$\frac{5}{16}$″—S	3	$\frac{1}{8}$	$\frac{1}{8}$	
$\frac{3}{8}$″—R	3	$\frac{1}{8}$	$\frac{1}{8}$	
$\frac{3}{8}$″—S	3	$\frac{1}{8}$	$\frac{3}{16}$	
$\frac{7}{16}$″—R	3	$\frac{1}{8}$	$\frac{3}{16}$	
$\frac{7}{16}$″—S	5	$\frac{1}{10}$	$\frac{1}{10}$	2 @ $\frac{1}{10}$
$\frac{1}{2}$″—R	5	$\frac{1}{10}$	$\frac{1}{10}$	2 @ $\frac{1}{10}$
$\frac{1}{2}$″—S	5	$\frac{1}{8}$	$\frac{1}{8}$	2 @ $\frac{1}{10}$
$\frac{9}{16}$″—R	5	$\frac{1}{8}$	$\frac{1}{8}$	2 @ $\frac{1}{10}$
$\frac{9}{16}$″—S	5	$\frac{1}{8}$	$\frac{1}{8}$	2 @ $\frac{1}{8}$
$\frac{5}{8}$″—R	5	$\frac{1}{8}$	$\frac{1}{8}$	2 @ $\frac{1}{8}$
$\frac{5}{8}$″—S	5	$\frac{1}{8}$	$\frac{3}{16}$	2 @ $\frac{1}{8}$
$\frac{11}{16}$″—R	5	$\frac{1}{8}$	$\frac{3}{16}$	2 @ $\frac{1}{8}$
$\frac{11}{16}$″—S	5	$\frac{1}{8}$	$\frac{1}{8}$	2 @ $\frac{3}{16}$
$\frac{3}{4}$″—R	5	$\frac{1}{8}$	$\frac{1}{8}$	2 @ $\frac{3}{16}$
$\frac{3}{4}$″—S	5	$\frac{1}{8}$	$\frac{3}{16}$	2 @ $\frac{3}{16}$
$\frac{13}{16}$″—R	5	$\frac{1}{8}$	$\frac{3}{16}$	2 @ $\frac{3}{16}$
$\frac{13}{16}$″—S	7	$\frac{1}{8}$	2 @ $\frac{1}{8}$	3 @ $\frac{1}{8}$
$\frac{7}{8}$″—R	7	$\frac{1}{8}$	2 @ $\frac{1}{8}$	3 @ $\frac{1}{8}$
$\frac{7}{8}$″—S	7	$\frac{1}{8}$	2 @ $\frac{5}{32}$	3 @ $\frac{1}{8}$
$\frac{15}{16}$″—R	7	$\frac{1}{8}$	2 @ $\frac{5}{32}$	3 @ $\frac{1}{8}$
$\frac{15}{16}$″—S	7	$\frac{1}{8}$	2 @ $\frac{3}{16}$	3 @ $\frac{1}{8}$
1″—R	7	$\frac{1}{8}$	2 @ $\frac{3}{16}$	3 @ $\frac{1}{8}$
1″—S	7	$\frac{1}{8}$	2 @ $\frac{1}{8}$	3 @ $\frac{3}{16}$
$1\frac{1}{16}$″—R	7	$\frac{1}{8}$	2 @ $\frac{1}{8}$	3 @ $\frac{3}{16}$
$1\frac{1}{16}$″—S	7	$\frac{1}{8}$	2 @ $\frac{1}{6}$	3 @ $\frac{3}{16}$
$1\frac{1}{8}$″—R	7	$\frac{1}{8}$	2 @ $\frac{1}{6}$	3 @ $\frac{3}{16}$
$1\frac{1}{8}$″—S	7	$\frac{1}{8}$	2 @ $\frac{3}{16}$	3 @ $\frac{3}{16}$
$1\frac{3}{16}$″—R	7	$\frac{1}{8}$	2 @ $\frac{3}{16}$	3 @ $\frac{3}{16}$
$1\frac{3}{16}$″—S	7	$\frac{1}{8}$	2 @ $\frac{7}{32}$	3 @ $\frac{3}{16}$

adhesive. Each class or type is divided into several different standard grades, and the rules for these grades are known as "Douglas Fir Plywood, Commercial Standard CS45–42." The description of each grade as set forth by the Douglas Fir Plywood Association is given in the following paragraphs.

MOISTURE-RESISTANT GRADES

These grades represent the majority of production and consist of plywood with a high degree of moisture resistance where its application requires that it shall retain its original form and practically all its strength when occasionally subjected to a thorough wetting and subsequent normal drying. Veneers $\frac{1}{12}$ in. or more shall be used in the construction of moisture-resistant type panels $\frac{1}{4}$ in. and upward in thickness. The veneer thickness shall be measured before the panel is sanded. This type is available in the following grades:

Sound 2 Sides (SO2S). Each face shall be of one or more pieces of firm, smoothly cut veneer. When of more than one piece, it shall be well joined and reasonably matched for grain and color at the joints. It shall be free from knots, splits, checks, pitch pockets, and other open defects. Streaks, discolorations, sapwood, shims, and neatly made patches shall be admitted. This grade shall present a smooth surface suitable for painting.

Sound 1 Side (SO1S). The face shall be of one or more pieces of firm, smoothly cut veneer. When of more than one piece, it shall be well joined and reasonably matched for grain and color at the joints. It shall be free from knots, splits, checks, pitch pockets, and other open defects. Streaks, discolorations, sapwood, shims, and neatly made patches shall be admitted. The face shall present a smooth surface suitable for painting. The back shall present a solid surface with all knots in excess of 1 in. patched and with the following permitted: not more than six knotholes or borer holes $\frac{5}{8}$ in. or less in greater dimension, and splits $\frac{1}{8}$ in. or less in width and pitch pockets not in excess of 1 in. in width or 3 in. in length or that do not penetrate through veneer to glue line. There may be any number of patches and plugs in the back.

Wallboard (WB). This is a 3-ply board of $\frac{1}{4}$-in. or $\frac{3}{8}$-in. sanded, or 5-ply $\frac{1}{2}$ in. sanded thickness, the face of which shall be of one or more pieces of firm, smoothly cut veneer. When of more than one piece it shall be well joined and reasonably matched for grain and color at the joints. It shall be free from knots, splits, pitch pockets, and other open defects. Streaks, discolorations, sapwood, shims, and neatly made patches shall be admitted. The face of this grade shall present a smooth surface suitable for painting. The back shall contain knotholes or pitch pockets, splits, and other defects in number and size that will not seriously affect the strength or serviceability of the panel and which cannot reasonably and economically be repaired to make a sound face. All wallboard panels shall be so designated by grade-marking each panel.

Sheathing (SH). This is an unsanded plywood made only in the following sizes: thicknesses $\frac{5}{16}$ in. and $\frac{3}{8}$ in. 3-ply; $\frac{1}{2}$ in. and $\frac{5}{8}$ in. 3- or 5-ply; widths

36 in. and 48 in.; length 96 in. The face shall present a solid surface except that the following will be permitted: (*a*) not more than ten knotholes none of which shall exceed 1½ in. with not more than five exceeding ¾ in. in greatest dimension; (*b*) no group of knotholes within any 12-in. diameter circle shall have an aggregate greatest dimension of more than 3 in.; (*c*) no splits wider than ⅛ in., nor any type of borer holes longer than 1 in., nor open pitch pockets more than 1 in. wide. There may be any number of patches and plugs in the face, but the face may not be of such quality that, if sanded, it will pass for a Wallboard face. No belt sanding is permissible. The back shall be at least equal in quality to a Wallboard grade back. No tape shall be permitted in the glue line. All sheathing panels shall be scored or marked for nailing to conform to standard spacing of lumber studding.

Industrial (Unsanded). Faces of panels shall be free from knotholes, and any type of borer holes more than ⅝ in. in greatest dimension and open pitch pockets more than 1 in. wide. Tight knots, checks, plugs, patches and shims shall be admitted in either face. Core and crossbands shall be of firm stock but shall contain no knotholes greater than 1¼ in. in any dimension.

Concrete-Form Plywood. Concrete-form plywood shall be built up of three or five thicknesses of veneer, of which the two outside plies are at least ⅛ in. thick before sanding, except for plywood ¼ in. in thickness. An occasional knothole is permissible in the center or core of 5-ply panels only, but no knotholes are permitted in cross banding. Appearance of faces shall be similar to that of "Sound 2 Sides" grade. The bonding agent used shall be especially prepared for this purpose and shall be very highly water-resistant. All concrete-form plywood shall be so designated by grade-marking each panel on the face. Concrete-form plywood shall be edge-sealed and shall have the faces mill-oiled unless the order specifically states not to oil.

EXTERIOR GRADES

These grades represent the ultimate in moisture resistance, a plywood that will retain its original form and strength when repeatedly wet and dried and otherwise subjected to the elements, and suitable for permanent exterior use. It shall be free from both core gaps and core voids that impair the strength or serviceability of the panel. Only a resin-impregnated tape shall be permitted in the glue line. No veneer thicker than 5/16 in. shall be used. All exterior panels shall be so designated by a distinctive symbol "Ext." branded or stamped on the edge of each panel. This type is available in the following grades:

Good 2 Sides Exterior (G2S-Ext.). Each face shall be of a single piece of smoothly cut veneer of 100 per cent heartwood, free from knots, splits, checks, pitch pockets, and other open defects. The faces shall be a yellow or pinkish color without stain. Shims that occur only at the ends of panels and inconspicuous well-matched small patches not to exceed ⅜ in. in width by 2½ in. in length shall be admitted. This grade is recommended for uses where a light stain or natural finish is desired.

Good 1 Side Exterior (G1S-Ext.). The face shall be equal to that described under "Good 2 Sides Exterior" grade, while the back shall be equal to the "Sound 2 Sides Exterior" grade.

Sound 2 Sides Exterior (SO2S-Ext.). Each face shall be of one or more pieces of firm, smoothly cut veneer. When of more than one piece, it shall be well joined and reasonably matched for grain and color at the joints. It shall be free from knots, splits, checks, pitch pockets, and other open defects. Streaks, discolorations, sapwood, shims, and neatly made patches shall be admitted. This grade shall present a smooth surface suitable for painting.

Sound 1 Side Exterior (SO1S-Ext.). The face shall be of one or more pieces of firm, smoothly cut veneer. When of more than one piece, it shall be well joined and reasonably matched for grain and color at the joints. It shall be free from knots, splits, pitch pockets, and other open defects. Streaks, discolorations, sapwood, shims, and neatly made patches shall be admitted. The face on this grade shall present a smooth surface suitable for painting. The back shall contain knotholes not larger than 1 in., splits not wider than 3⁄16 in., and other defects in number and size that will not impair the serviceability of the panel and that cannot be reasonably and economically repaired to make a sound face.

Sheathing Exterior (SH-Ext.). An unsanded panel, the face of which shall present a solid surface except that the following will be permitted: (*a*) not more than six knotholes 3⁄8 in. or less in greatest dimensions; (*b*) splits 1⁄16 in. or less in width; (*c*) one or two strips of paper tape. There may be any number of patches and plugs in the face but the face may not be of such quality that, if sanded, it will pass for "Sound 1 Side Exterior" grade. No belt sanding is permissible. The back shall be the same as the back described under "Sound 1 Side Exterior" grade.

Industrial Exterior. Shall have two solid faces made of one or more pieces. All open defects shall be repaired, except small pitch pockets and tight splits which are 1⁄16 in. or under in width. All knotholes in the face veneer shall be patched. Panels in this grade shall be lightly "touch" sanded on both sides to remove dry tape, surplus glue, etc., but the tolerance of 1⁄32 in., as allowed for unsanded panels, shall apply.

Concrete-Form Exterior. Shall be the same as "Sound 2 Sides Exterior" except that faces shall be 1⁄8 in. thick before sanding. It is made only in 5⁄8 in. and 3⁄4 in. thicknesses. All concrete-form plywood shall be so designated by grade-marking each panel on the face. Concrete-form plywood shall be edge-sealed and shall have the faces mill-oiled unless the order specifically states not to oil.

195. Design Stresses. The following tables on design methods and allowable stresses are taken from the Forest Products Laboratory Bulletin, "Approximate Tentative Methods of Calculating the Strength of Plywood," by L. J. Markwardt, Principal Engineer, and A. D. Freas, Assistant Engineer.

Table 50. Design Method and Allowable Stresses for Douglas Fir Plywood [1]

Property	Direction of Stress with Respect to Direction of Face Grain	Portion of Cross-Sectional Area to be Considered	Unit Stress to be Used
Tension	Parallel or perpendicular	Parallel plies [2] only	Unit stress for extreme fiber in bending.
	±45°	Full cross-sectional area	One-fourth unit stress for extreme fiber in bending.
Compression	Parallel or perpendicular	Parallel plies [2] only	Unit stress in compression parallel to grain.
	±45°	Full cross-sectional area	One-third unit stress in compression parallel to grain.
Shear	Parallel or perpendicular	Full cross-sectional area	Double unit stress for horizontal shear.
	±45°	Full cross-sectional area	Four times unit stress for horizontal shear.
Shear in Plane of Plies	Parallel, perpendicular, or ±45°	Full shear area	1. Conditions without stress concentration: *A.* Joints between plies in plywood panels acting as a beam—one-half unit stress for horizontal shear. 2. Conditions with stress concentration: *A.* Symmetrical concentration. Joints in panels with stressed plywood covers, for interior joists with end headers, or without end headers if ratio of joist depth to joist width does not exceed 2—one-half unit stress for horizontal shear. *B.* Unsymmetrical concentration. Joints in I and box beams with plywood webs; and joints in panels with stressed plywood covers, for *exterior* joists with end headers, or without end headers if ratio of joist depth to joist width does not exceed 2—one-fourth unit stress for horizontal shear.
Load in Bending	Parallel or perpendicular	Bending moment $M = KSI/c$, where S = unit stress for extreme fiber in bending; I = moment of inertia computed on basis of parallel plies only; c = distance from neutral axis to outer fiber of outermost ply having its grain in the direction of the span; $K = 1.50$ for three-ply plywood having the grain of the outer plies perpendicular to the span; $K = 0.85$ for all other plywood.	Unit stress for extreme fiber in bending.
Deflection in Bending	Parallel or perpendicular	Deflection may be calculated by the usual formulas, taking as the moment of inertia that of the parallel plies plus $\frac{1}{20}$ that of the perpendicular plies. (When face plies are parallel, the calculation may be simplified, with but little error, by taking the moment of inertia as that of the parallel plies only.)	Unit value for modulus of elasticity.
Deformation in Tension or Compression	Parallel or perpendicular	Parallel plies [2] only	Unit value for modulus of elasticity.
Bearing at Right angles to Plane of Plywood		Loaded area	Unit stress in compression perpendicular to grain.

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[1] The suggested simplified methods of calculation in this table apply reasonably well with usual plywood types under ordinary conditions of service. It is recognized, however, that they are not entirely valid for all types of plywood and plywood constructions, or for all spans and span-depth ratios.

[2] By "parallel plies" is meant those plies whose grain direction is parallel to the direction of principal stress.

Appropriate unit stress for Table 50 may be obtained from Table 51 by multiplying the basic values by the appropriate reduction factor, as follows:

1. For Douglas fir plywood to be used in dry locations (moisture content 16 per cent or less), the basic stresses for extreme fiber in *bending, compression perpendicular to grain,* and *compression parallel to grain* may be increased by 25 per cent. (No increase for maximum horizontal shear or modulus of elasticity.)

2. The basic stresses are for clear wood without defects. An appropriate reduction factor is to be used according to the estimated grade of material with respect to defects allowed. When defects present are estimated to reduce the strength one-fourth, multiply the basic stresses by three-fourths, etc.

EXAMPLE. What unit stress should be used for tension parallel to face grain for plywood of a three-fourths grade to be used in dry locations?

Procedure: $2000 \times 1.25 \times 0.75 = 1875$ lb. per sq. in.

For 5-ply panels in compression parallel to the grain of the face plies, the unit stress should be taken as that for the next lower grade.

196. Strength Properties. In computing the tensile or compressive strength of Douglas fir plywood it is customary to consider only those plies having their grain parallel to the direction of the load. For example, consider a ¼-in. (SO2S) 3-ply panel that is to be used in a dry inside location.

$$\text{Thickness of face} = \tfrac{1}{9} \text{ in.}$$

$$\text{Thickness of core} = \tfrac{1}{9} \text{ in.}$$

$$\text{Thickness of back} = \tfrac{1}{9} \text{ in.}$$

Table 51. Basic Stresses for Douglas Fir

	Pounds per Square Inch
Extreme fiber in bending	2,000
Compression perpendicular to grain	325
Compression parallel to grain	1,466
Maximum horizontal shear	120
Modulus of elasticity	1,600,000

Table 52. Example of Unit Stresses for Douglas Fir Plywood of Three-Fourths Grade to be Used in a Dry Inside Location

Property	*Direction of Stress with Respect to Face Grain*	*Unit Stress (Pounds per Square Inch)*
Tension	Parallel or perpendicular	1,875
Tension	±45°	470
Compression	Parallel or perpendicular	1,375
Compression	±45°	460
Shear	Parallel or perpendicular	180
Shear	±45°	360
Shear in plane of plies		22 [1]
Bending	Parallel or perpendicular	1,875
Modulus of elasticity	Parallel or perpendicular	1,600,000
Compression perpendicular to grain (bearing)		400

[1] Applies to shear stresses, such as in I-beams and box beams, where the stress is concentrated at the inner edge of the flange. For other conditions as outlined in Table 50, different values would be found as calculated by the appropriate factor.

Tension, allowable load per inch of width,

T_L,* grain of face plies parallel to load $= 2 \times \frac{1}{9} \times 2190 = 487$ lb.

T_T,* grain of face plies perpendicular to load $= 1 \times \frac{1}{9} \times 2190 = 243$ lb.

Compression, allowable load per inch of width,

$$C_L = 2 \times \tfrac{1}{9} \times 1605 = 356 \text{ lb.}$$

$$C_T = 1 \times \tfrac{1}{9} \times 1605 = 178 \text{ lb.}$$

Moment of inertia, for a strip 1 in. wide,

$$I_L = \tfrac{1}{12}[(\tfrac{1}{4})^3 - (\tfrac{1}{9})^3] = 0.00118$$

$$I_T = \tfrac{1}{12}(\tfrac{1}{9})^3 = 0.00017$$

Bending, allowable bending moment per inch of width,

$$M_L = \frac{0.85SI}{c} = \frac{0.85 \times 2190 \times 0.00118}{\frac{1}{8}} = 17.6 \text{ in.-lb.}$$

$$M_T = \frac{1.5SI}{c} = \frac{1.5 \times 2190 \times 0.00017}{\frac{1}{18}} = 10.05 \text{ in.-lb.}$$

* Subscripts L and T refer to the direction of the grain of the face plies. L denotes the face grains parallel to the applied load or parallel to the span in bending. T is the opposite condition with the face grains perpendicular to the applied load.

Table 53. Working Stresses for Douglas Fir Plywood, Damp or Wet Location

[Moisture content of the plywood under load may exceed 16 per cent. Technical Data on Plywood, Douglas Fir Plywood Association, 1942]

	(These two exterior grades available only on special mill order)		87½% SO2S	80% Plywall or SO1S	75% Sheathing
	Clear 100% G2S	95% G1S			
Extreme fiber in bending	2,000	1,900	1,750	1,600	1,500
Compression perpendicular to grain	325	325	325	325	325
Compression parallel to grain	1,466	1,390	1,285	1,170	1,100
Maximum horizontal shear	120	114	105	96	90
Modulus of elasticity	1,600,000	1,600,000	1,600,000	1,600,000	1,600,000

Table 54. Working Stresses for Douglas Fir Plywood, Dry Location

[Moisture content does not exceed 16 per cent. Technical Data on Plywood Douglas Fir Plywood Association, 1942]

	Clear 100% G2S	95% G1S	87½% SO2S	80% Plywall or SO1S	75% or Plyscord Sheathing
Extreme fiber in bending	2,500	2,375	2,190	2,000	1,875
Compression perpendicular to grain	405	405	405	405	405
Compression parallel to grain	1,830	1,735	1,605	1,460	1,375
Maximum horizontal shear	120	114	105	96	90
Modulus of elasticity	1,600,000	1,600,000	1,600,000	1,600,000	1,600,000

Deflection, allowable deflection for a strip 1 in. wide with a load of 40 lb. per sq. ft. over a span of 12 in.,

$$Y_L = \frac{5}{384} \times \frac{40}{12} \times \frac{12 \times 12 \times 12}{1,600,000 \times 0.00118} = 0.0396 \text{ in.}$$

$$Y_T = \frac{5}{384} \times \frac{40}{12} \times \frac{12 \times 12 \times 12}{1,600,000\left(0.00017 + \dfrac{0.00118}{20}\right)} = 0.264 \text{ in.}$$

197. Gusset Plates. Plywood panels thicker than those customarily manufactured offer a convenient means of forming timber joints when bolts are used alone or with timber connectors. Several investigators have conducted tests on Douglas fir plywood to ascertain the safe working loads of connectors when plywood is used in wood to wood joints.* Although these tests have not covered all possible joint assemblies, certain conclusions can be made with respect to the design loads for connectors when used with plywood. The minimum recommended thicknesses of Douglas fir plywood should be the same as those recommended for Douglas fir lumber, and the standard design loads for connectors should be taken as the same as those for Douglas fir when the load is at an angle of 60° to the grain. These recommendations are based on the results of tests by Prof. Williams and are fairly well substantiated by the tests conducted on 2½-in. split rings by Messrs. Benkert, Hanrahan, and Smith. They recommend, however, that the standard edge distance be increased for plywood. A minimum edge distance for the 2½ rings should be 3 in. while the minimum spacing and end distance should be 5 in.

198. Plywood Girders. The use of plywood for the webs of large wooden I-beams is a comparatively recent development. The usual construction is a plywood web ⅜ to ¾ in. thick and either solid or glued laminated flanges and stiffeners. The flanges and stiffeners may be glued or bolted to the web. Sufficient test data are lacking to provide an exact theory for use in design, but the following method is suggested as the method likely to give results well within a safe limit.

Consider the girder as shown in Fig. 129, and determine the safe uniform load per foot that it is capable of carrying. The web is ⅜ in. 3-ply Douglas fir plywood with the face veneers ⅛ in. thick and the core 3⁄16 in. The flanges and stiffeners are to be Douglas fir glued to the web with casein glue.

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* "Laboratory Tests on Structural Plywood," by H. A. Williams, *Eng. News-Record,* June 16, 1938. "Plywood for Gusset Plates," by H. N. Benkert, F. J. Hanrahan, and L. W. Smith, *Eng. News-Record,* June 23, 1938.

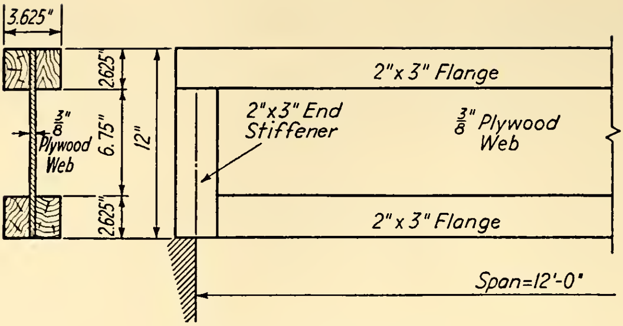

FIG. 129. Plywood girder.

Design stresses, plywood with face grains in direction of span:

Extreme fiber in bending = 2500 lb. per sq. in.
Compression perpendicular to grain = 405 lb. per sq. in.
Compression parallel to grain = 1830 lb. per sq. in.
Maximum horizontal shear = 240 lb. per sq. in.
Shear in plane of plies = 30 lb. per sq. in.
Modulus of elasticity = 1,600,000 lb. per sq. in.

Design stresses, solid wood:

Extreme fiber in bending = 1100 lb. per sq. in.
Compression perpendicular to grain = 390 lb. per sq. in.
Compression parallel to grain = 1075 lb. per sq. in.
Maximum horizontal shear = 110 lb. per sq. in.
Modulus of elasticity = 1,600,000 lb. per sq. in.

In determining the load capacity of a plywood girder there are three main things to consider. These are (1) the flanges must resist the moment due to the load, (2) the glue joint between the web and the flange must be sufficient to resist the horizontal shear along this line, and (3) the web must be stiffened to resist the buckling loads caused by the vertical loads and reactions.

Moment of inertia and statical moment of cross-sectional area about neutral axis: In computing I and Q, only the plies having their grain parallel to the span are considered.

$$I = \frac{2 \times 3.25 \times 2.625^3}{12} + 2 \times 3.25 \times 2.625 \times 4.688^2 + \frac{2}{8} \times \frac{12^3}{12}$$

$$= 421.75 \text{ in.}^4$$

$$Q = 3.25 \times 2.625 \times 4.688 + \tfrac{2}{8} \times 6 \times 3 = 44.5 \text{ in.}^3$$

Flexure: The usual formula for flexure must be multiplied by the appropriate form factor F so that the fiber stress $f = FMc/I$. The form factor may be calculated as follows (see Chapter IV):

$$F = 0.58 + 0.42\left[K\left(\frac{t_2 - t_1}{t_2}\right) + \frac{t_1}{t_2}\right]$$

K = depends on ratio d_F/D

d_F = depth of compression flange

D = total depth

t_2 = width of flange

t_1 = thickness of web

Table 55. Values of K for Various Values of d_F/D

$\frac{d_F}{D}$	K	$\frac{d_F}{D}$	K	$\frac{d_F}{D}$	K
0.10	0.085	0.45	0.660	0.80	0.985
0.15	0.155	0.50	0.740	0.85	0.995
0.20	0.230	0.55	0.810	0.90	0.998
0.25	0.315	0.60	0.875	0.95	1.000
0.30	0.400	0.65	0.920	1.00	1.000
0.35	0.490	0.70	0.950		
0.40	0.575	0.75	0.970		

$$F = 0.58 + 0.42\left[0.264\left(\frac{3.625 - 0.375}{3.265}\right) + \frac{0.375}{3.625}\right] = 0.72$$

$$f = \frac{FMc}{I} \qquad M = \frac{fI}{Fc} = \frac{1100 \times 421.75}{0.72 \times 6} = 107{,}500 \text{ in.-lb.}$$

$$\frac{w \times 12 \times 12 \times 12}{8} = 107{,}500 \qquad w = 496 \text{ lb. per ft.}$$

Shear between flange and web:

$$v = \frac{VQ}{Id_1} \qquad V = \text{vertical shear} = \frac{206 \times 12}{2} = 1236 \text{ lb.}$$

$$d_1 = \text{length of contact} = 2.625 \times 2 = 5.25$$

$$v = \frac{1236 \times 44.5}{421.75 \times 5.25} = 25 \text{ lb. per sq. in.}$$

The allowable unit stress is 30 lb. per sq. in.

End stiffeners: The end stiffeners prevent the web from buckling under the concentrated end reaction and should be designed to carry the entire load without aid from the web. The area in bearing between the end stiffeners and the top flange must be great enough that the unit stress in compression perpendicular to the grain is not exceeded.

$$V = \frac{206 \times 12}{2} = 1236 \text{ lb.}$$

Unit stress in compression parallel to grain $= \dfrac{1236}{2 \times 1.625 \times 1.312} =$ 290 lb. per sq. in.

Allowable unit stress = 1075 lb. per sq. in.

Unit stress in bearing perpendicular to grain of top flange $= \dfrac{1236}{2 \times 1.625 \times 2.625} = 145$ lb. per sq. in.

Allowable unit stress = 390 lb. per sq. in.

Horizontal shear: Based on a load of 206 lb. per ft., the vertical shear = 1236 lb.

The unit shearing stress in the plywood web $= \dfrac{1236 \times 44.5}{421.75 \times \frac{3}{8}} =$ 348 lb. per sq. in.

The maximum horizontal shear is 240 lb. per sq. in., and the allowable load must be reduced to 142 lb. per ft.

If the face plies had their grain at an angle of 45° to the axis of the beam and the grain of the core was perpendicular to the grain of the face plies, the allowable unit horizontal shear would be 480 lb. per sq. in.

Web buckling: At any point in the web the horizontal and vertical unit shears are equal and produce an equal resultant in compression which acts at an angle of 45° to the edge of the web, provided that it is assumed that the web does not resist any part of the stresses due to bending. This compressive stress must be kept small enough to prevent the web from buckling or stiffeners must be added to reinforce the web.

The unit shearing stress at which buckling will occur may be expressed by the following formula: *

$$S_{cr} = \frac{\pi^2 E t^2 K}{12 b^2 (1 - V^2)}$$

* *Strength of Materials,* S. Timoshenko, Part 2.

where E = modulus of elasticity at 45°.

t = thickness of web.

b = smallest dimension.

V = Poisson's ratio.

K = a constant depending upon the equation $\left(\frac{a}{nb} + \frac{nb}{a}\right)^2$.

a = largest dimension.

n = an integer representing the number of waves into which the panel will buckle.

Table 56. Values of K in Terms of a and b

a/b	1	1.2	1.4	1.5	1.6	1.8	2.0	2.5	3	∞
K	9.42	8	7.3	7.1	7.0	6.8	6.6	6.3	6.1	5.4

The value of Poisson's ratio for most plywood is not well known, but for practical purposes it may be taken as zero. We do know that it will be less than unity, and by assuming a value of zero a factor of safety is introduced. The modulus of elasticity for Douglas fir plywood at 45° to the grain may be taken as 260,000.

With no stiffeners, a = 141.375 and b = 6.75

$$\frac{a}{b} = \frac{141.375}{6.75} = 21 \qquad K = 5.4$$

$$S_{cr} = \frac{3.14^2 \times 260{,}000 \times 0.375^2 \times 5.4}{12 \times 6.75^2} = 3570 \text{ lb. per sq. in.}$$

Introducing a factor of safety of 5, S_{cr} = 714 lb. per sq. in.

As the above shearing stress which will cause the web to buckle is much greater than the allowable horizontal shear, no stiffeners other than the one at each end are required.

If the grain of the plies makes an angle of 45° with the axis of the beam, the modulus of elasticity is taken parallel to the face plies and becomes 1,600,000 lb. per sq. in. In this case the shearing stress required to cause buckling would be increased considerably.

A more accurate method * of determining the critical shearing stress is to consider the elastic constants of the plywood and determine the value of the constants D_1, D_2, and D_3 for use in the following formulas. For a simply supported rectangular plate

$$\theta = \frac{\sqrt{D_1 D_2}}{D_3} \qquad B = \frac{b}{a}\sqrt[4]{\frac{D_1}{D_2}}$$

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* *Theory of Elastic Stability*, S. Timoshenko, 1936.

and the critical value of the shearing force when $\theta > 1$ may be obtained from the formula

$$F_{cr} = \frac{4K\sqrt[4]{D_1 D_2^3}}{b^2}$$

K is a factor depending on θ and B, which can be taken from Chart 37. This chart is the same as that given in "The Critical Shear Load of Rectangular Plates," by E. Seydel, Tech. Memo. 705, N.A.C.A.

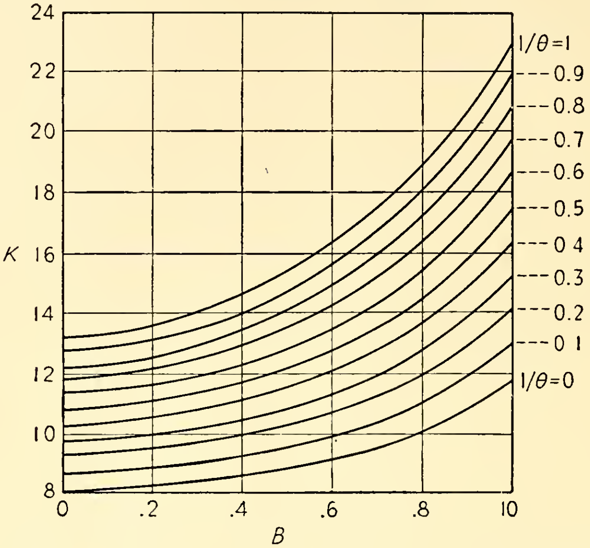

CHART 37. Values of K for use in the formula for determining the critical buckling stress due to shearing stresses in plywood plates.

When $\theta < 1$ the critical value of the shearing force may be found from the formula

$$F_{cr} = \frac{4K\sqrt{D_2 D_3}}{b^2}$$

Table 57. Values of K for an Infinitely Long Plate

θ	0	0.2	0.5	1.0
K	11.7	11.8	12.2	13.17

The above equations can be used for long plates with clamped edges when K is taken from Table 58.

Table 58. Values of K for an Infinitely Long Plate with Clamped Edges

θ	0	0.2	0.5	1	2	3	5	10	20	40	∞
K	18.6	18.9	19.9	22.15	18.8	17.6	16.6	15.9	15.5	15.3	15.1

For plywood with flat-grained veneers *

$$D_1 = \frac{E_1 h^3}{12V} \qquad D_2 = \frac{E_2 h^3}{12V} \qquad D_3 = \frac{E_L V_{TL} h^3}{12V} + \frac{G_{TL} h^3}{6}$$

where E_1 = modulus of elasticity of the entire panel taken in a direction.

E_2 = modulus of elasticity of the entire panel taken in b direction.

h = thickness of panel.

V = $1 - V_{LT}V_{TL}$, where V_{LT} and V_{TL} are Poisson's ratios. Their product = 0.01.

E_L = modulus of elasticity along the grain, 1,600,000.

V_{TL} = Poisson's ratio for extension along the grain and contraction tangent to growth rings = 0.019.

G_{TL} = modulus of rigidity in shear = $E_L/16$.

E_T = modulus of rigidity across the grain = 1,600,000 × 0.045 = 72,000.

a = largest dimension.

b = smallest dimension.

The exact modulus of elasticity of plywood is the summation of the product of the moment of inertia of each ply about the center of the panel and its modulus of elasticity, divided by the moment of inertia of the entire cross section.

For a 3-ply panel with all the plies of the same thickness, the modulus of elasticity can be reduced to a simple form in terms of E_L and E_T.

Let T be the total thickness

$$E_1 = \frac{\left[\frac{1}{12} \times 2 \times \left(\frac{T}{3}\right)^3 + 2 \times \frac{T}{3} \times \left(\frac{T}{3}\right)^2\right] E_L + \left[\frac{1}{12} \times \left(\frac{T}{3}\right)^3\right] E_T}{\frac{T^3}{12}}$$

$$E_1 = \tfrac{1}{27}(26E_L + E_T)$$

$$E_2 = \frac{\left[\frac{1}{12} \times 2 \times \left(\frac{T}{3}\right)^3 + 2 \times \frac{T}{3} \times \left(\frac{T}{3}\right)^2\right] E_T + \left[\frac{1}{12} \times \left(\frac{T}{3}\right)^3\right] E_L}{\frac{T^3}{12}}$$

$$E_2 = \tfrac{1}{27}(E_L + 26E_T)$$

---

* For the derivation of the constants D_1, D_2, and D_3 see Article 204.

For the $\frac{3}{8}$-in. 3-ply web under consideration the plies are not all of the same thickness. The face and back are $\frac{1}{8}$ in. and the core is $\frac{3}{16}$ in. This makes a total thickness of $\frac{7}{16}$ in.

$$E_1 = 1{,}485{,}000 \qquad D_1 = 10{,}410$$

$$E_2 = 193{,}000 \qquad D_2 = 1{,}358$$

$$D_3 = 1{,}273$$

$$\theta = \frac{\sqrt{D_1 D_2}}{D_3} = 2.945 \qquad B = \frac{b}{a}\sqrt[4]{\frac{D_1}{D_2}} = 0.08$$

From Chart 37, $K = 9.8$

$$F_{cr} = \frac{4K\sqrt[4]{D_1 D_2^3}}{b^2} = \frac{4 \times 9.8\sqrt[4]{10{,}410 \times 1358^3}}{6.75 \times 6.75} = 1940$$

$$S_{cr} = \frac{1940 \times 16}{7} = 4430 \text{ lb. per sq. in.}$$

Introducing a factor of safety of 5, $S_{cr} = 886$ lb. per sq. in.

The load-carrying capacity of the girder is limited by the working stress in horizontal shear.

AIRCRAFT PLYWOODS

199. General. Plywood for aircraft is manufactured under very exacting conditions, and only a limited number of species are suitable for its manufacture. However, the success with which plywood has been used in aircraft construction demonstrates the ability of manufacturers to produce it with the precision commensurate with design methods. This ability may be attributed to the knowledge of the characteristics and strength properties of the various species and the methods of handling veneers throughout the entire manufacturing process.

The chief advantages of plywood for aircraft construction are its high ratio of strength to weight, the ease with which it may be repaired, its relatively low cost, and its extreme adaptability for a number of parts.

Plywood in aircraft construction is subjected to a combination of stresses that are seldom found, or designed for, in large engineering structures. Under combined loading the stress is not proportional to the load, and for this reason airplane parts are designed on the basis of ultimate stresses.

200. Species. The Army-Navy aeronautical specifications, AN-NN-P-511a, contain the species of wood which may be used in plywood construction. The species are divided into three groups as shown in Table 59.

Table 59. Permissible Species of Wood for Aircraft Plywood Construction

Group 1	*Group 2*	*Group 3*
Beech, American	Birch, Alaska, white, and paper	Basswood, American
Birch, sweet and yellow	Mahogany, African	Poplar, yellow
Maple, hard	Mahogany, Central American	Cedar, Port Orford
	Maple, soft	Spruce, Sitka, red, and white (quarter-sliced)
	Gum, red and tupelo	
	Walnut, black	
	Douglas fir (quarter-sliced)	

Group 1 contains species with high strength properties, hardness, and resistance to abrasion. The species in Groups 1 and 2 take the best finish and are best suited for the manufacture of curved parts. Group 3 woods are used primarily for core stock, but they do have high bending and buckling resistance on the basis of the strength-weight ratio.

Beech, American. Beech ranks favorably with sweet and yellow birch and hard maple in strength properties. However, it is slightly heavier and is not readily available in the high grades.

Birch, Sweet and Yellow. Sweet and yellow birch have the same general properties. They are hard and stiff and rather heavy. However, their fine texture makes them capable of taking a high finish. They are used primarily for propellers and veneer for plywood.

Maple, Hard. Hard maple is dense, hard, and stiff and has a fine even texture. It is superior in hardness to all other species used in aircraft. It is used for propellers, veneer for plywood, and bearing blocks.

Mahogany. African mahogany is lighter than mahogany from Central America, but in most strength properties the two species are very similar.

Maple, Soft. Soft maple is much lower in weight and strength than hard maple. However, it is suitable for plywood where a semihard species is required.

Gum. Red gum is heavier than spruce and has higher strength properties with the exception of stiffness. It has a tendency to warp, but through careful manufacture it is a good species for plywood.

Tupelo gum is considerably heavier than spruce, but it is lower in stiffness. It can be used in plywood but is not adaptable for other aircraft uses.

Walnut, Black. Black walnut is probably the best wood for propeller construction. It has good hardness to resist wear and has the ability to retain its shape under varying moisture conditions. Its fine texture and finishing qualities make it ideal for instrument panels and cabinets.

Douglas Fir (Coast Region). Douglas fir from the West Coast is considerably heavier than spruce and has strength properties exceeding those of spruce. Because of its tendency to splinter, considerably more care is necessary in the manufacture of Douglas fir than in most of the other species. Quarter-sliced Douglas fir is suitable for aircraft plywood.

Basswood, American. Basswood is a light-weight wood, but it is low in most strength properties. However, it works easily and can be nailed without splitting. It is a desirable wood for use in aircraft as veneer for plywood, wing ribs, and webs.

Poplar, Yellow. Yellow poplar is not much heavier than spruce, but some of its strength properties are slightly less than those of spruce. It is easily worked, retains its shape, and in some respects is a good substitute for spruce. It has been used extensively in aircraft plywood.

Cedar, Port Orford. Port Orford cedar is somewhat heavier than spruce, with strength properties slightly higher. It has not been used a great deal, but it is satisfactory for plywood.

Spruce, Sitka, Red, and White. All three species of spruce have the same strength properties. However, Sitka spruce is more abundant and is available in larger sizes. For these reasons Sitka spruce is used more often than red and white spruces. The excellent strength properties of spruce combined with a high ratio of strength to weight make it an ideal material for wing beams, struts, ribs, webs, landing gear, stiffeners, flooring, longérons, and veneer for plywood. Spruce is considered the standard of comparison for the suitability of other species for aircraft construction.

201. Sizes. Plywood panels for aircraft construction are made in a number of thicknesses and combinations of veneers. The required veneer thickness and species for each nominal thickness are given in the specifications AN-NN-P-511a and are reproduced here.

202. Design Values for Species Used in Aircraft Plywood. Table 61 presents strength data on various species for use in aircraft design. The values are based on a moisture content of 15 per cent and a duration of stress of 3 seconds. They apply only to material that meets the minimum specific gravity requirements and the limitations as to defects. The allowable defects have been set forth in Report 354 of the National Advisory Committee for Aeronautics and reprinted in Bulletin 1079 of the United States Department of Agriculture.

Table 60. Plywood Panel Construction

			Thickness of Plies											
			Group I or II Faces with Group III Inner Plies				All Group I or all Group II				All Group III			
			F&B	XB	IB	C	F&B	XB	IB	C	F&B	XB	IB	C
			Inch	Inch	Inch	Inch	Inch	Inch	Inch	Inch	Inch	Inch	Inch	Inch
1/32	3	0.031 ± 0.004	......	......	......	......	1/100[a]	......	......	1/64[a]	......	......	......	......
3/64	3	.047 ± .006	......	......	......	......	1/64[a]	......	......	1/48[a]	......	......	......	......
1/16	3	.063 ± .007	1/48	......	......	1/32	1/48	......	......	1/32	1/48	......	......	1/28
5/64	3	.078 ± .008	1/48	......	......	1/24	1/48	......	......	1/24	1/48	......	......	1/20
3/32	3	.094 ± .009	1/32	......	......	1/24	1/32	......	......	1/24	1/32	......	......	1/20
1/8	3	.125 ± .010	1/28	......	......	1/16	1/28	......	......	1/16	1/28	......	......	1/14
5/32	3	.157 ± .011	1/24	......	......	1/12	1/24	......	......	1/12	1/24	......	......	1/12
5/32	5	.157 ± .011	1/32	1/24	......	1/32	1/32	1/24	......	1/32	1/32	1/24	......	1/32
3/16	3	.188 ± .012	1/20	......	......	1/10	1/20	......	......	1/10	1/20	......	......	1/10
3/16	5	.188 ± .012	1/28	1/20	......	1/28	1/28	1/20	......	1/32	1/28	1/20	......	1/28
7/32	5	.219 ± .013	1/24	1/16	......	1/32	1/24	1/16	......	1/32	1/24	1/16	......	1/28
1/4	5	.250 ± .014	1/20	1/16	......	1/20	1/20	1/16	......	1/24	1/20	1/16	......	1/20
5/16	5	.313 ± .016	1/16	1/12	......	1/20	1/16	1/12	......	1/24	1/16	1/12	......	1/20
3/8	5	.375 ± .018	1/16	1/10	......	1/12	1/16	1/10	......	1/12	1/16	1/10	......	1/12
7/16	7	.438 ± .020	1/16	1/16	1/16	1/10	1/16	1/16	1/16	1/10	1/16	1/16	1/16	1/10
1/2	7	.500 ± .022	1/16	1/10	1/14	1/14	1/16	1/10	1/14	1/16	1/16	1/10	1/14	1/14
9/16	7	.563 ± .023	1/16	1/10	1/10	1/12	1/16	1/10	1/10	1/12	1/16	1/10	1/10	1/12
5/8	9	.625 ± .025	1/16[b]	1/16[b]	1/14	......	1/16	1/16	1/16	......	1/16	1/16	1/14	......
				1/10	......	1/12	......	1/10	......	1/10	......	1/10	......	1/12
3/4	9	.750 ± .028	1/12[b]	1/12[b]	1/10	......	1/12	1/12	1/10	......	1/12	1/12	1/10	......
				1/10	......	1/12	......	1/10	......	1/14	......	1/10	......	1/10
7/8	11	.875 ± .031	1/12[b]	1/12[b]	1/12	......	1/12	......	1/14	......	1/12	1/12	1/12	......
				1/10	1/12	1/12	......	1/10	1/12	1/12	......	1/10	1/12	1/10
1	11	1.000 ± .035	1/10[b]	1/10[b]	1/10	......	1/10	1/10	1/10	......	1/10	1/10	1/10	......
				1/10	1/10	1/10	......	1/10	1/10	1/10	.....	1/10	1/10	1/10

[a] All plies of plywood thinner than 1/16 in. are of same species.

[b] Faces and outer cross bands of plywood having 9 or more plies are of same species.

F&B = Face and back.
XB = Cross bands.
IB = Inner bands.
C = Core.

Table 61. Design Values for Species Used in Aircraft Construction

[*Wood in Aircraft Construction*, by G. W. Trayer, National Lumber Manufacturers' Association, 1930]

Species	Minimum Specific Gravity Based on Volume and Weight Oven-Dry	Static Bending				Compression Parallel to Grain		Compression Perpendicular to Grain	Shearing Strength Parallel to Grain	Hardness, Side; Load Required to Imbed 0.444 In. Ball ½ Its Diam.
		Fiber Stress Elastic Limit Lb./Sq. In.	Modulus of Rupture Lb./Sq. In.	Modulus of Elasticity 1000 Lb./Sq. In.	Work to Maximum Load In. Lb./Cu. In.	Fiber Stress at Elastic Limit Lb./Sq. In.	Maximum Crushing Strength Lb./Sq. In.	Lb./Sq. In.	Lb./Sq. In.	Lb.
Basswood	0.36	5,600	8,600	1,250	6.6	3,370	4,500	620	720	370
Beech	.60	8,200	14,200	1,440	13.5	4,880	6,500	1,670	1,300	1,060
Birch (sweet and yellow)	.58	9,500	15,500	1,780	18.2	5,480	7,300	1,590	1,300	1,100
Cedar, Port Orford	.40	7,400	11,000	1,520	8.7	4,880	6,100	1,030	760	520
Douglas fir (coast region)	.45	8,000	11,500	1,700	8.1	5,600	7,000	1,300	810	620
Gum, red	.48	7,500	11,600	1,290	10.0	4,050	5,400	1,190	1,100	650
Gum, tupelo	.47	6,700	9,300	1,090	6.3	3,980	5,300	1,400	1,080	80
Magnolia, southern	.48	6,350	10,800	1,210	11.8	3,680	4,900	1,390	1,040	920
Mahogany, African	.42	7,900	10,800	1,280	8.0	4,280	5,700	1,400	980	720
Mahogany, Central American	.46	8,800	11,600	1,260	7.3	4,880	6,500	1,760	860	790
Maple, soft	.45	5,800	8,600	985	7.6	3,520	4,700	1,190	1,000	630
Maple, hard	.60	9,500	15,000	1,600	13.7	5,620	7,500	2,170	1,520	1,270
Spruce, red, white, and Sitka	.36	6,200	9,400	1,300	7.8	4,000	5,000	840	750	440
Walnut, black	.52	10,200	15,100	1,490	11.4	5,700	7,600	1,730	1,000	990
Yellow poplar	.38	6,000	9,100	1,300	6.5	3,750	5,000	810	800	420

NOTE: The above values are for use in Aircraft Design and are based on a duration of stress of 3 sec. and a moisture content of 15%. The true modulus of elasticity in bending, E_{TR}, may be obtained by adding 10 per cent to the modulus of elasticity given above. G, the modulus of rigidity in shear, may be taken as $E_{TR}/16$.

203. Plywood Strength Properties. The strength properties of plywood are calculated on the basis of reducing proportionally the veneer thickness so that the summation of the thickness of all veneers equals the nominal plywood thickness. The methods for calculating strength properties of plywood given in Table 50 are approximate, and, although they are very useful for obtaining estimates of the strength of aircraft plywood, they will not give results which are comparable to those obtained by more exact methods.

STATIC BENDING

Modulus of elasticity: The modulus of elasticity of a plywood panel is a summation of the product of the modulus of elasticity of each ply and its moment of inertia about the neutral axis divided by the moment of inertia of the geometrical cross section of the panel. The general formula for E_1 and E_2 is written

$$E_1 = \frac{1}{I} \sum_{i=1}^{i=n} E_i I_i$$

where E_1 = modulus of elasticity measured parallel to the grain of the face plies.

E_2 = modulus of elasticity measured perpendicular to the grain of the face plies.

The modulus of elasticity for the various species may be taken from Table 60. These values are in the direction of or parallel to the grain. For values across the grain multiply the modulus of elasticity along the grain by 0.045.

EXAMPLE. Consider a $\frac{3}{16}$-in. 3-ply birch panel 1 in. wide.

$E_L =$	1,780,000	Face	$= \frac{1}{10}$ in.
$E_T =$	80,200	Core	$= \frac{1}{20}$ in.
$I =$	$\frac{1}{12}(\frac{3}{16})^3$	Back	$= \frac{1}{10}$ in.
		Nominal thickness	$= \frac{3}{16}$ in.

Adjusted values:

Face $= \frac{3}{64}$ in.
Core $= \frac{3}{32}$ in.
Back $= \frac{3}{64}$ in.

$$E_1 = \frac{1{,}780{,}000 \times 2[\frac{1}{12}(\frac{3}{64})^3 + \frac{3}{64}(\frac{9}{128})^2] + 80{,}200[\frac{1}{12}(\frac{3}{32})^3]}{\frac{1}{12} \times (\frac{3}{16})^3}$$

$$E_1 = 1{,}567{,}000$$

$$E_2 = \frac{80{,}200 \times 2[\frac{1}{12}(\frac{3}{64})^3 + \frac{3}{64}(\frac{9}{128})^2] + 1{,}780{,}000[\frac{1}{12}(\frac{3}{32})^3]}{\frac{1}{12}(\frac{3}{16})^3}$$

$$E_2 = 293{,}000$$

Stiffness factor: The stiffness factor is the product of the modulus of elasticity and the moment of inertia.

$$E_1I = 1{,}567{,}000 \times \tfrac{1}{12}(\tfrac{3}{16})^3 = 860.0$$

$$E_2I = 293{,}000 \times \tfrac{1}{12}(\tfrac{3}{16})^3 = 160.8$$

Bending: Values for moment at proportional limit are the bending moments that cause 1-in. plywood strips to reach their proportional limit.

M_L = moment when grain of face plies is parallel to the direction of the span

M_T = moment when grain of face plies is perpendicular to the direction of the span

$$M_L = \frac{E_1I \times f}{C \times E_L} \times 0.85$$

$$M_T = \frac{E_2I \times f}{C \times E_L} \times 0.90$$

where E_1I = stiffness factor.
E_2I = stiffness factor.
f = fiber stress at proportional limit for solid wood from Table 61.
C = distance from neutral axis to extreme fiber of outermost longitudinal ply.
E_L = modulus of elasticity of outermost longitudinal ply.

Values for the moments at modulus of rupture are those moments which will cause the panel to fail. The formulas given above apply except that the modulus of rupture for solid wood is used for *f*.

EXAMPLE. 3/16-in. 3-ply birch panel 1 in. wide.

Moment at proportional limit:

$$M_L = \frac{860 \times 9500 \times 0.85}{\frac{3}{32} \times 1{,}780{,}000} = 41.50$$

$$M_T = \frac{160.8 \times 9500 \times 0.90}{\frac{3}{64} \times 1{,}780{,}000} = 16.50$$

Moment at failure:

$$M_L = \frac{860 \times 15{,}500 \times 0.85}{\frac{3}{32} \times 1{,}780{,}000} = 67.80$$

$$M_T = \frac{160.8 \times 15{,}500 \times 0.90}{\frac{3}{64} \times 1{,}780{,}000} = 26.90$$

TENSION

The allowable ultimate tensile stress for plywood when the face grain is parallel to the load is equal to the sum of the strengths of the longitudinal plies divided by the total area of the cross section.

EXAMPLE. $\frac{3}{16}$-in. 3-ply birch panel.

Face grains parallel to load:

$$\text{Ultimate tensile strength} = \frac{15{,}500 \times 2 \times \frac{3}{64}}{\frac{3}{16}} = 7750 \text{ lb. per sq. in.}$$

$\text{Face} = \frac{1}{10}$ in.

$\text{Core} = \frac{1}{20}$ in.

$\text{Back} = \frac{1}{10}$ in.

Adjusted values:

$T = \frac{3}{16}$ in.

$\text{Face} = \frac{3}{64}$ in.

$\text{Core} = \frac{3}{32}$ in.

$\text{Back} = \frac{3}{64}$ in.

COMPRESSION

Modulus of elasticity:

E_1 = modulus of elasticity when face grain is parallel to the direction of force

E_2 = modulus of elasticity when face grain is perpendicular to the direction of force

E_1 and E_2 are a summation of the product of the modulus of elasticity and the cross-sectional area of each ply divided by the total cross-sectional area. The modulus of elasticity of each ply should be taken as 10 per cent greater than the values given in Table 46.

EXAMPLE. $\frac{3}{16}$-in. 3-ply birch panel 1 in. wide.

$$E_1 = \frac{1{,}958{,}000 \times \frac{3}{32} + 88{,}000 \times \frac{3}{32}}{\frac{3}{16}} = 1023$$

$$E_2 = \frac{88{,}000 \times \frac{3}{32} + 1{,}958{,}000 \times \frac{3}{32}}{\frac{3}{16}} = 1023$$

Fiber stress at proportional limit:

F_L = fiber stress when face grain is parallel to load

F_T = fiber stress when face grain is perpendicular to load

$$F_L = \frac{E_1 \times f_2}{E_L}$$

$$F_T = \frac{E_2 \times f_2}{E_L}$$

where f_2 = fiber stress at proportional limit for solid wood in compression parallel to grain.

E_L = modulus of elasticity along the grain.

Both values are from Table 61.

3/16-in. 3-ply birch panel 1 in. wide.

$$F_L = \frac{1023 \times 5480}{1{,}780{,}000} = 3150 = F_T \text{ for this particular panel}$$

Maximum crushing strength:

$$F_L = \frac{E_1 \times f_3}{E_L}$$

$$F_T = \frac{E_2 \times f_3}{E_L}$$

$$F_L = \frac{1023 \times 7300}{1{,}780{,}000} = 4200$$

where f_3 = maximum crushing strength for solid wood in compression parallel to grain (from Table 61).

204. Bending and Buckling of Plywood Plates. The general equation for the bending of plywood plates can be derived in the same manner as the equation for isotropic materials.* It is necessary, of course, to use different constants in setting up the equations, and the derivation is based on the assumption that cross sections remain plane during bending and rotate only with respect to their neutral axes.

Consider a panel subjected to uniformly distributed bending moments along its sides, as shown in Fig. 130.* The plane midway between the faces is taken as the *xy*-plane with the *z*-axis perpendicular to it. In Fig. 131 * is shown an element cut from the plate.

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* *Theory of Elastic Stability*, S. Timoshenko, 1936.

M_x = bending moment per unit length of edges parallel to y-axis
M_y = bending moment per unit length of edges parallel to x-axis
h = total thickness
$1/r_x$ = curvature of neutral section parallel to zx-plane
$1/r_y$ = curvature of neutral section parallel to zy-plane

The unit elongation of the element *abcd* in the x and y directions at a distance z from the neutral surface may be represented as

$$\epsilon_x = \frac{z}{r_x} \qquad \epsilon_y = \frac{z}{r_y}$$

From Hooke's law:

$$\epsilon_x = \frac{S_x}{E_x} - \frac{V_{yx}S_y}{E_y} \qquad \epsilon_y = \frac{S_y}{E_y} - \frac{V_{xy}S_x}{E_x}$$

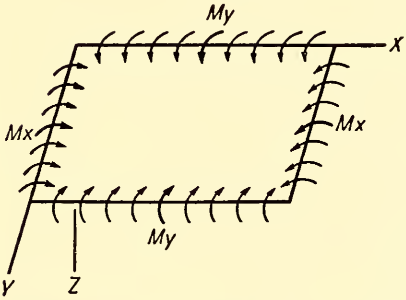

Fig. 130. Plate subjected to bending moments.

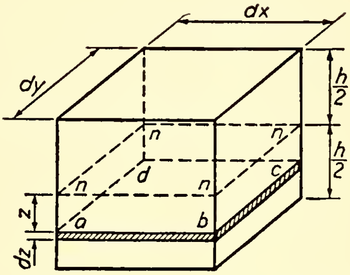

Fig. 131. Element cut from plate.

From the above equations, letting $1 - V_{yx}V_{xy} = V$ gives

$$S_x = \frac{E_xZ}{V}\left(\frac{1}{r_x} + V_{yx}\cdot\frac{1}{r_y}\right)$$

$$S_y = \frac{E_yZ}{V}\left(\frac{1}{r_y} + V_{xy}\cdot\frac{1}{r_x}\right)$$

where S_x and S_y are the stresses in the element *abcd.*

These normal stresses produce couples which must be equal to the external moments.

$$M_x\,dy = \int_{-h/2}^{h/2} S_xZ\,dy\,dz$$

$$M_y\,dx = \int_{-h/2}^{h/2} S_yZ\,dx\,dz$$

Substituting the values for S_x and S_y and integrating gives

$$M_x = \frac{E_x}{V}\left(\frac{1}{r_x} + V_{yx} \cdot \frac{1}{r_y}\right)\frac{h^3}{12}$$

$$M_y = \frac{E_y}{V}\left(\frac{1}{r_y} + V_{xy} \cdot \frac{1}{r_x}\right)\frac{h^3}{12}$$

If we denote the deflection by w, the formulas for the curvature of the plate may be expressed in terms of w.

$$\frac{1}{r_x} = -\frac{d^2w}{dx^2} \qquad \frac{1}{r_y} = -\frac{d^2w}{dy^2}$$

Substituting these values in the equations for M_x and M_y, we have

$$M_x = -\frac{E_x h^3}{12V}\left(\frac{d^2w}{dx^2} + V_{yx}\frac{d^2w}{dy^2}\right)$$

$$M_y = -\frac{E_y h^3}{12V}\left(\frac{d^2w}{dy^2} + V_{xy}\frac{d^2w}{dx^2}\right)$$

The twisting moments M_{xy} and M_{yx} may be derived; they have the following values:

$$M_{xy} = -M_{yx} = \frac{G_{xy}h^3}{6}\frac{d^2w}{dx\,dy}$$

For a distributed lateral load acting perpendicular to the middle plane of the plate, the vertical shearing forces must be considered. These will have the following values:

$$P_x = \int_{-h/2}^{h/2} q_x\,dz \qquad P_y = \int_{-h/2}^{h/2} q_y\,dz$$

For equilibrium of the forces acting on the element it is necessary to consider the projections of all forces on the z-axis and the moments of all forces with respect to the x- and y-axes.

Projection of forces on the z-axis

$$\frac{dP_x}{dx}\,dx\,dy + \frac{dP_y}{dy}\,dy\,dx + W\,dx\,dy = 0$$

where W is the load per unit area on the face of the plate.

Simplified, the above formula becomes

$$\frac{dP_x}{dx} + \frac{dP_y}{dy} + W = 0$$

Moments of all forces acting on *abcd* with respect to the x-axis are

$$\frac{dM_{xy}}{dx} - \frac{dM_y}{dy} + P_y = 0$$

Moments of all forces acting on *abcd* with respect to the y-axis are

$$\frac{dM_{yx}}{dy} - \frac{dM_x}{dx} - P_x = 0$$

By substitution

$$\frac{d^2M_x}{dx^2} + \frac{d^2M_{yx}}{dx\,dy} + \frac{d^2M_y}{dy^2} - \frac{d^2M_{xy}}{dx\,dy} = -W$$

$$M_{yx} = -M_{xy}$$

$$\frac{d^2M_x}{dx^2} - 2\frac{d^2M_{xy}}{dx\,dy} + \frac{d^2M_y}{dy^2} = -W$$

This is the same derivation as that given by Professor Timoshenko.

Substituting the values obtained for M_x, M_y, and M_{xy} in the above equations, we obtain

$$D_1\frac{d^4w}{dx^4} + B\frac{d^4w}{dx^2\,dy^2} + 2K\frac{d^4w}{dx^2\,dy^2} + D_2\frac{d^4w}{dy^4} + C\frac{d^4w}{dy^2\,dx^2} = W$$

where

$$D_1 = \frac{E_xh^3}{12V} \qquad B = D_1V_{yx} \qquad D_2 = \frac{E_yh^3}{12V} \qquad C = D_2V_{xy} \qquad K = \frac{G_{xy}h^3}{6}$$

This equation can be further simplified by letting

$$D_3 = \frac{B + C + 2K}{2}$$

$$D_1\frac{d^4w}{dx^4} + 2D_3\frac{d^4w}{dx^2\,dy^2} + D_2\frac{d^4w}{dy^4} = W$$

This is the general equation for the deflection surface of a plywood plate.

For the solution of this equation refer to Mimeographs 1300 and 1316 of the Forest Products Laboratory.

Approximate formulas for deflections and the stresses developed have been derived for rectangular plywood panels which have the face-grain direction parallel or perpendicular to the edges. These are presented in the ANC Handbook on the Design of Wood Aircraft Structures, July 1942.

For plywood made with flat-grained or rotary-cut veneers,

$$D_1 = \frac{E_1 h^3}{12V} \qquad D_2 = \frac{E_2 h^3}{12V}$$

$$\frac{E_x V_{yx}}{V} = \frac{E_y V_{xy}}{V} = \frac{E_L V_{TL}}{V}$$

$$D_3 = \left(\frac{E_L V_{TL}}{V}\frac{h^3}{12} + E_L V_{TL}\frac{h^3}{12} + 2G_{TL}\frac{h^3}{6}\right)\frac{1}{2}$$

$$D_3 = \frac{h^3}{12V}\left(E_L V_{TL} + 2G_{TL}V\right)$$

The critical buckling stress for a plate subjected to compression on two opposite sides may be found by substituting the above values for D_1, D_2, and D_3 in Professor Timoshenko's formula.

$$S_{cr} = \frac{\pi^2}{b^2 h}\left(D_1\frac{b^2}{a^2} + 2D_3 + D_2\frac{a^2}{b^2}\right)$$

$$S_{cr} = \frac{\pi^2}{b^2 h}\cdot\frac{h^3}{12V}\left(E_1\frac{b^2}{a^2} + 2E_L V_{TL} + 4G_{TL}V + E_2\frac{a^2}{b^2}\right)$$

$$S_{cr} = \frac{\pi^2 h^2}{12 b^2 V}\left(E_1\frac{b^2}{a^2} + 2E_L V_{TL} + 4G_{TL}V + E_2\frac{a^2}{b^2}\right)$$

The smallest value of the critical stress is obtained when

$$\frac{a}{b} = \left(\frac{D_1}{D_2}\right)^{\frac{1}{4}} = \left(\frac{E_1}{E_2}\right)^{\frac{1}{4}}$$

$$S_{cr} = \frac{2\pi^2}{b^2 h}\left(\sqrt{D_1 D_2} + D_3\right)$$

where a = unloaded side in compression and largest dimension in shear.
b = loaded side in compression and smallest dimension in shear.
E_1 = modulus of elasticity in a direction.
E_2 = modulus of elasticity in b direction.

205. Box Beams. In designing box beams with plywood webs it is necessary to multiply the fiber stress by the appropriate form factor as explained in Article 63.

Shear, actual: The maximum shear stresses in the plywood webs may be computed by the following formula:

$$q = \frac{VQ}{It}$$

where q = shear stress in pounds per square inches.
V = external shear.
Q = statical moment of area above or below neutral axis.
I = moment of inertia.
t = combined thickness of webs.

Shear, allowable: The allowable shear stresses of plywood webs having the face-grain direction at 0°, 45°, or 90° to the main beam axis may be obtained from Chart 38.

a = clear distance between flanges or clear distance between stiffeners, whichever is smaller

$$a_0 = t\left(K_s \frac{E_L}{F_{s\theta}}\right)^{1/2}$$

where t = total thickness of webs in inches.

$$K_s = \text{shear constant} = \frac{K_1}{3}\left[\left(\frac{E_1}{E_L}\right)^3 \frac{E_2}{E_L}\right]^{1/4}$$

where K_1 = constant obtained from Chart 39.
$F_{s\theta}$ = allowable (ultimate) stress for plywood in shear where θ is the angle of the face plies from the horizontal.

$$F_{s\theta} = \frac{1}{\sqrt{\left(\frac{1}{(F_{tu})_a{}^2} + \frac{1}{(F_{cu})_b{}^2}\right)\sin^2 2\theta + \frac{\operatorname{Cos}^2 2\theta}{F_{su}{}^2}}}$$

where $(F_{tu})_a$ = ultimate tensile strength of plywood with grain of face plies parallel to the stress.
$(F_{cu})_b$ = ultimate compressive strength of plywood with grain of face plies perpendicular to the stress.
F_{su} = ultimate shear stress of the plywood when the face grain is parallel and perpendicular to the shear stresses.

The allowable ultimate shear stress F_{su} is equal to the sum of the longitudinal shear strengths of all the plies divided by the total area. The shear strength of any ply is obtained by multiplying 1.33 times the

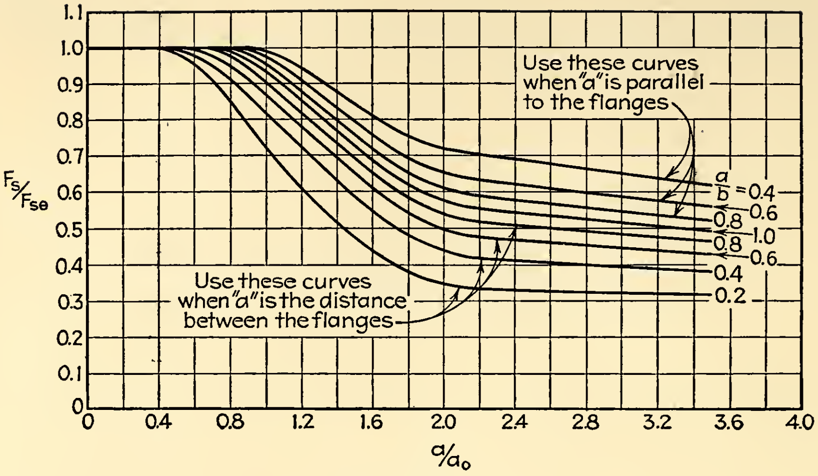

Chart 38.

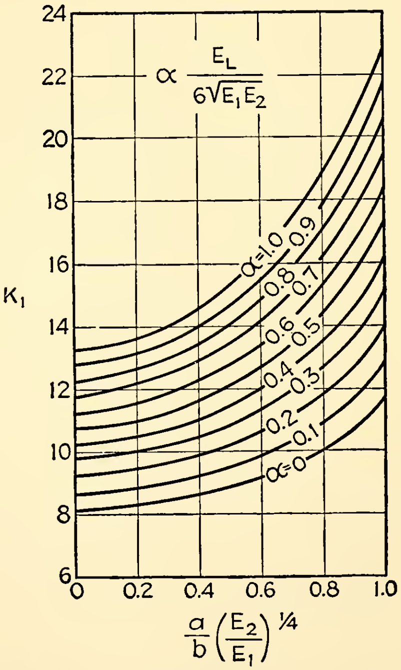

Chart 39.

allowable shear stress parallel to the grain (Table 61) by the area of that ply.

The value of $F_{s\theta}$ given previously is based on θ being positive or measured counterclockwise from the horizontal to the grain direction for θ between 0° to 90°. When θ is negative or between 0° and −90°, $F_s\theta$ becomes

$$F_{s\theta} = \frac{1}{\sqrt{\left(\frac{1}{(F_{cu})_a{}^2} + \frac{1}{(F_{tu})_b{}^2}\right)\sin^2 2\theta + \frac{\text{Cos}^2 2\theta}{F_{su}{}^2}}}$$

The ultimate tensile strength $(F_{tu})_a$ or $(F_{tu})_b$ is equal to the sum of the strengths of the longitudinal plies divided by the total area of the cross section. The strength of any longitudinal ply is equal to its area multiplied by the modulus of rupture for the species of that ply given in Table 61.

The ultimate compressive strength $(F_{cu})_a$ or $(F_{cu})_b$ may be obtained from the following formulas:

$$(F_{cu})_a = E_a\left(\frac{F_{cu}}{E_L}\right)$$

$$(F_{cu})_b = E_b\left(\frac{F_{cu}}{E_L}\right)$$

$$E_a = \frac{1}{A}\sum_{i=1}^{i=n} E_iA_i$$

where A = total cross-sectional area.
A_i = area of ith ply.
E_i = modulus of elasticity of ith ply measured in same direction as E_a.
F_{cu} = ultimate crushing stress from Table 61.

PROBLEMS

1. Find the load that a ¾-in. Douglas fir plywood strip 1 in. wide can carry in tension and compression. Use grade SO2S in a dry inside location. Consider the face plies parallel to the direction of the load.

2. For the same size and grade of Douglas fir plywood as that in the previous problem find the allowable bending moment. Determine the deflection over a span of 2 ft. for the load that produces the allowable bending moment. Consider the face plies parallel to the direction of the span.

3. A 1-in. rough Douglas fir plywood panel is used as a sub-flooring over joists spaced 28 in. on center. The load is 40 lb. per sq. ft. Calculate the deflection when the face plies are parallel to the span and a three-fourths grade is used.

4. Plywood girders spaced 12 ft. apart support a uniformly distributed load of 40 lb. per sq. ft. over a span of 24 ft. Using select structural Douglas fir flanges and stiffeners and an SO2S grade of plywood for the web, design the girder.

5. A plywood girder spans 36 ft. and carries a load of 400 lb. per ft. If select structural Douglas fir is used in the flanges and stiffeners, which are attached to the web by means of split-ring connectors and bolts and a G2S grade of plywood is used in the web, what size girder is required, and how many bolts and split rings are needed?

6. For a 5/8-in. 9-ply all birch panel compute the stiffness factor, the moment at the proportional limit, and the modulus of elasticity in compression. Assume a strip 1 in. wide and calculate the above properties for both directions of the face grain.

7. Compute the modulus of elasticity in static bending and the ultimate tensile strength for a 7/32-in. 5-ply panel with birch faces and poplar cross bands and core.

8. A 3/8-in. 5-ply all birch panel is 30 in. wide and 60 in. long. If a load in compression is applied across the 30-in. width find the critical buckling stress.

9. What is the critical buckling stress for the panel in problem 6 if the panel is 30 in. wide and 90 in. high and load in compression is applied across the 30-in. width.

10. Locate the neutral axis of the box beam shown in Fig. 132, and determine the internal resisting moment and the allowable vertical shear. The webs are mahogany plywood and the flanges are spruce.

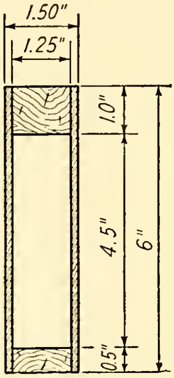

FIG. 132. Box beam.

CHAPTER XI

DECAY, WOOD-DESTROYING ORGANISMS, AND PRESERVATIVES

DECAY

206. General. Unless wood is given an approved preservative treatment or unless it contains a high percentage of naturally durable heartwood it will decay when used under high humidity conditions. Excessive dampness encourages the growth of wood-destroying fungi, which usually appear as lacelike plants on the surface of wood. These plants require food, moisture, air, and a favorable temperature in order to live, and most species of wood contain the required food supply in their cells or cell walls. Wood with a moisture content above the fiber saturation point is particularly susceptible to decay by fungi, but wood that is continuously soaked in water or continuously dry will not decay.

207. Stains and Molds. Wood-destroying fungi in the early stages of their growth produce stains or molds that are not particularly injurious to the wood provided the wood does not remain in a damp location. Little discoloration of the wood is caused by molds, and these can be easily brushed or surfaced off. Fungi cause stains which usually appear as spots or streaks on the surface of wood, the most common of which is known as blue stain. If the wood is not permitted to remain under improper moisture and temperature conditions, blue stain will not develop into a stage of decay; therefore it is not objectionable where appearance is not the governing factor.

208. Prevention of Decay. A general precaution to prevent decay in building construction is to use seasoned lumber and to provide suitable ventilation and construction features so that no condensation of moisture will take place on the wood. The roofs of weave sheds, bleacheries, and other factories where high humidities exist should be provided with an efficient moisture barrier beneath the roof rafters so that moisture will be kept from the upper part of the roof where it will condense.

Because laminated floors, built-up beams, and columns of unseasoned lumber cannot dry out properly when put in place they are apt to decay.

Foundations for buildings which enclose the entire structure should have openings to provide proper ventilation under the house to prevent decay. The usual method is to use screened openings with an area totaling about 3 per cent of the house area.

Wood posts should not bear directly on a basement floor nor should they be embedded directly into the concrete. Instead, concrete base blocks extending above the floor surface should be used to provide the proper drainage and protection against fungus.

During construction, lumber should be properly stacked and covered to protect it from rain. The building itself may get wet, but the surface wetting of the open framing will dry off quickly.

Window and door frames should be pitched away from the building so that no moisture can accumulate, and drainage outlets and ventilation should be provided under all outside columns.

WOOD-DESTROYING ORGANISMS

209. Marine Borers. There are two main classes of marine borers—the molluscan and the crustacean. Although both groups are found in salt or brackish water, their methods of attack on wood are somewhat different. Molluscans usually enter the wood when they are very small, leaving a small entrance hole, and then grow larger as they burrow into the wood. The crustaceans attack the wood from the outside, so the destruction is more readily measured by visual inspection than that by the molluscan group.

The molluscan group includes several species of *Teredo, Bankia,* and *Martesia,* and the crustacean group includes *Limoria, Chelura,* and *Sphaeroma.*

No untreated wood is immune to the attack of marine borers, and although most borers exist only in brackish waters, some of the molluscan group have been found in fresh water in Australia, India, and parts of South America.

210. Termites. There are two types of termites, (1) the ground-inhabiting or subterranean termites, and (2) the dry-wood termites. Both resemble ants somewhat in size, general appearance, and living habits and for this reason are often called "white ants." The subterranean termites are found in nearly every state in this country and do most of the damage to wood structures. The dry-wood termites are found along a narrow strip of territory from central California along the southern boundary of the United States to Virginia. The subterranean termites require moisture to sustain life, so they must have access to the ground at all times. Consequently, they build tunnels through the earth and around obstructions in order to get at their chief source of food supply, which is the cellulose in wood. The dry-wood termites are able to live in wood without contact with the ground, and, although they do only a small fraction of the total destruction by termites, they are a definite menace in some regions.

Very few species of wood are capable of resisting attack by termites. California redwood and southern cypress when used above ground will offer some resistance, and the resinous heartwood from southern pine is rather resistant, but it is not available in quantities large enough to be of importance in the construction industry.

211. Protection of Wood. The only chemical method of protecting wood from attack by marine borers is to use a heavy treatment of an approved preservative. The preservative should penetrate into the piece as deeply as possible, and the wood should absorb a high percentage of the preservative.

When wood is in contact with the ground, preservative treatments are also recommended. Often it is impossible or impracticable to treat all wood used above the ground that might be subject to termite attack. The best protection here is to construct the building in such a manner that the subterranean termites cannot gain access to it. A number of precautions or methods of construction to avoid termite attack are enumerated below.*

1. Construct foundations of brick, stone, concrete, or fully treated lumber piling. If a mortar is used, a cement mortar is preferable. If there is a basement, it should be floored with concrete.
2. Posts that support the first-floor beams should not rest on the ground or wood blocking unless it is thoroughly treated. They should be supported by concrete piers extending above the basement floor.
3. Avoid dampness, warmth, and darkness by providing ventilation under the building.
4. Construct concrete floors on a gravel base to prevent dampness and cracking.
5. Do not leave stakes or forms embedded in the concrete or next to concrete foundations because they will afford an easy means for the termites to enter the building.
6. Remove all debris from under the building, whether above or below the ground.
7. Foundations under the building should extend 12 to 18 in. above the ground. The exterior grade line may be 6 in. from the closest wood if the foundation can be readily inspected for termite tubes.
8. Place termite shields, which are sheets of galvanized iron, in the surface of the masonry, and let them project horizontally for 2 in. and

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* "Preventing Damage by Termites or White Ants," by T. E. Snyder, *Bull.* 1472, U. S. Dept. Agr.

then downward at an angle of 45° for 2 in. These make it impossible for termites to build their tubes up the side of the foundation and into the wood. Place circular metal shields projecting 3 in. on all sides of pipes under the building.

Paint is a good protection against dry-wood termites, and all exposed portions of a building in a locality where dry-wood termites are prevalent should be adequately painted. Interior unpainted portions may be protected by placing fine screens over all openings.

PRESERVATIVES

212. General. The use of suitable preservatives can greatly extend the life of wood that is used under conditions favoring decay or attack by insects or marine borers. Some preservatives are more effective than others, and the adoption of any particular type depends upon the ultimate use of the lumber. In general, a preservative is a chemical of high toxicity. An effective preservative must remain chemically stable, have a high degree of permanence, possess good penetrating properties, and be harmless to wood as well as metal. Other requirements, depending upon the use for which the lumber is intended, may be cleanness, cheapness, paintability, freedom from odor, and fire resistance.

Wood preservatives may be divided into three general classes: (1) toxic oils, which are relatively insoluble in water; (2) salts injected into the wood in the form of water solutions; and (3) small percentages of a toxic material combined with a solvent other than water.

213. Creosote. There are many types of creosote preservatives, but of all of them coal-tar creosote is the most important and most generally useful. It is a black or dark brown oil which is made by the distillation of coal tar. It has high toxicity, is relatively insoluble in water, is easy to apply, and is generally available at low cost. As yet no better preservative has been developed for treating timbers for general outdoor use, but for some purposes coal-tar creosote has disadvantages. It has an objectionable odor, and when freshly applied it will ignite easily. Moreover, creosoted wood cannot be painted and is not clean to handle.

Wood-tar creosote has not been used extensively enough to compare it with coal-tar creosote, but it is generally believed to be effective when properly prepared and injected.

Water-gas-tar creosote is made by the distillation of water-gas-tar and is, therefore, a petroleum product. It is not as toxic or as effective as coal-tar creosote but is considered a good preservative.

Mixtures of coal-tar with coal-tar creosote are often used for the treatment of ties and to some extent for other purposes. The addition of the coal-tar reduces the cost of the preservative; but the solutions do not penetrate as easily as the straight coal-tar creosote, and they make the timber considerably dirtier to handle.

The addition of petroleum to coal-tar creosote makes a solution that is used extensively for the treatment of railroad ties. This solution does not penetrate as readily and is less toxic than the straight coal-tar creosote, but actual records of its use have demonstrated that it has similar decay resistance.

214. Water-Soluble Salts. A number of water-soluble salts, including zinc chloride, chromated zinc chloride, sodium fluoride, arsenic, copper sulphate, and mercuric chloride, are used as wood preservatives. Of all these, zinc chloride has been most widely used in the United States. Its chief advantages are cleanliness, paintability, lack of fire hazard, lack of odor, general availability, and relatively low cost. Its main disadvantage, which is common to all water-soluble salts, is its tendency to leach out of the wood when it is in contact with water or soil.

Chromated zinc chloride is rapidly taking the place of zinc chloride as the leading water-soluble salt preservative. It contains 18½ per cent sodium dichromate, and the remaining percentage is the standard zinc chloride. Tests indicate that it is superior to zinc chloride.

Sodium fluoride has been used to some extent in this country for treating mine timbers and ties, and when mixed with other materials it has been used for treating structural timbers. It seems to be a good preservative, but it has not been used extensively enough for comparison with other salt treatments.

Arsenic has been used for several years in treating poles and ties. Used either alone or mixed with other materials, it has good preservative qualities. However, no definite long-time records are available as to its effectiveness.

Copper sulphate has not been used to any extent in the United States. It tends to corrode steel, and its use requires special treating equipment.

215. Toxic Materials Dissolved in Solvents Other Than Water. In the past several years a number of preservatives have been developed that may be applied without heat. Usually all that is necessary is to soak the wood in the preservative. These preservatives are colorless, odorless, clean, and paintable. They do not require a great deal of time to dry, and they will not swell the wood to any appreciable extent. They are used primarily for the treatment of window frames and sash, doors, flooring, and millwork.

Preservatives of this type that are commercially available contain a toxic chemical usually in the form of pentachlorphenol or tetrachlorphenol and a volatile solvent such as solvent naphtha. These preservatives have demonstrated their effectiveness as a treatment for window sash and frames, and they may prove suitable for many other uses.

216. Methods of Treatment. There are two general methods of treating wood. These are the pressure and nonpressure processes. The most effective treatment is either the full-cell or the empty-cell pressure process. In both cases the timber to be treated is stacked on steel cars and moved into a large cylinder. The cylinder is then closed and filled with a preservative, and pressure is applied to force the preservative into the wood.

The full-cell or Bethel process is used primarily when the maximum amount of retention of oil preservatives is desired or when a limited absorption of preservatives in water solution is specified. A full-cell treatment with creosote oils will provide wood with a high degree of protection. Zinc chloride, chromated zinc chloride, and sodium fluoride are usually applied by this process. After the timber has been placed in the cylinder a vacuum is applied to remove air from the cylinder and the wood. The preservative is then admitted to the cylinder without breaking the vacuum. Pressure is applied until the desired absorption is obtained. The pressure and the length of time required to complete the treatment depend upon the ease with which the wood takes the preservative. The preservative is then drawn from the cylinder and a final vacuum created to free the wood from dripping preservative.

The empty-cell process is used when it is desired to obtain a deep penetration but low net retention of preservative. No initial vacuum is used, and compressed air or air at atmospheric pressure is trapped in the wood cells when the preservative is applied under pressure. The pressure is continued until the wood refuses to take the preservative or until the desired penetration is reached. Then the entrapped air forces part of the preservative out of the wood.

There are several ways of applying preservatives without pressure, but none are as effective as the pressure processes. The most common and the one most nearly approaching the pressure processes in thoroughness of treatment is known as the hot and cold bath treatment. It consists of immersing the wood in a hot bath of preservative in an open tank and then allowing the preservative to cool. The hot bath forces some of the air and moisture out of the wood, and during the cooling period the preservative is sucked into the wood to replace the air. The length of the hot and cold bath depends upon the ease with which the wood takes treatment, but it is usually several hours. For well-seasoned wood that takes preservative readily a hot bath of 2 or 3 hours and a cold bath of the same duration are usually sufficient. The temperature of the hot bath varies depending upon the type of preservative. For water solutions temperatures between 170° and 200° F. are employed, and for creosote treatments the temperatures used are between 200° and 230° F. The temperature of the cold bath should be sufficient to keep creosote preservatives in a fluid condition, usually 100° to 120° F.

Other nonpressure treatments are known as the steeping and dipping processes. In the steeping process the wood is simply soaked in unheated preservative, usually mercuric chloride, for a week or more. The dipping process consists in submerging the wood in a heated preservative from 5 to 15 minutes. The penetration obtained in these processes is very small, usually about $\frac{1}{16}$ in.

217. Specifications. The American Wood Preservers' Association has prepared specifications covering the details of various treating processes. These are in use by the Federal Government and have been adopted with slight revisions by the American Railway Engineering Association, the American Society for Testing Materials, and the American Society of Civil Engineers.

The A.W.P.A. specifications for No. 1 creosote oil for ties, piles, posts, structural timber, or butt treatment of poles are as follows:

> The creosote oil shall be a distillate of coal-gas tar or coke-oven tar and shall comply with the following requirements:
>
> 1. It shall contain not more than 3 per cent of water.
> 2. It shall contain not more than 0.5 per cent of matter insoluble in benzol.
> 3. The oil shall yield not more than 2 per cent coke residue.
> 4. The specific gravity at 38° C. compared with water at 15.5° C. shall not be less than 1.03.
> 5. The distillate, based on water-free oil, shall be within the following limits:
>
> Up to 210° C. not more than 5 per cent.
> Up to 235° C. not more than 25 per cent.
>
> 6. The tests shall be made in accordance with the standard methods of the American Wood Preservers' Association.

The A.W.P.A. specifications for zinc chloride are as follows:

> The zinc chloride shall be acid free and shall not contain more than 0.1 per cent iron. Fused or solid zinc chloride shall contain at least 94 per cent chloride of zinc. Concentrated zinc chloride solution shall contain at least 50 per cent chloride of zinc.

Table 62 is a schedule of recommended practice for the preservative treatment of timber for various uses. The Federal Government specifications and the corresponding American Wood Preservers' Association specifications are given. These specifications do not provide for treatment by nonpressure processes.

Table 62. Specifications for the Preservative Treatment of Wood

[Federal Specifications TT-W-571b]

Preservative Oils

Form of product and service	Minimum net retention of preservative recommended			Treating specification (the current issue of A. W. P. A. standard specification listed)
	Coal-tar creosote (current issue of Fed. Spec. TT-W-556, A. W. P. A. Spec. 4)	Creosote-coal-tar solution (current issue of Fed. Spec. TT-W-566, A. W. P. A. Spec. 5e)	Creosote-petroleum solution (current issue of Fed. Spec. TT-W-568, A. W. P. A. Spec.)	
	Lb. per cu. ft.	*Lb. per cu. ft.*	*Lb. per cu. ft.*	
Cross ties, switch ties, bridge ties:				
Douglas fir, hemlocks, lodgepole pine	7	7	8	Use Douglas Fir Specification 38.
Oak, red, sweet gum (red gum), and black gum	7	7	8	Use Oak Specification 52.
Pine, southern yellow, ponderosa, red, jack	7	7	8	Use Southern Yellow Pine Specification 53.
Other species	7	7	8	No specification available.
Piles, for land or fresh water use:				
Douglas fir	10	10	12	41.
Oak, red and white	10	10		Use Douglas fir Specification 41.
Pine, lodgepole	10	10	12	Do.
red	12	12		Use Southern Yellow Pine Specification 39.
southern yellow	12	12		39.
Piles, marine use:				
Douglas fir	14	14		41.
Pine, southern yellow	20	20		39.
Poles:				
Cedar (butt treatment, incised)				43.
Cedar (full-length, pressure treatment with or without incision).	4			
Douglas fir	8			51.
Pine, lodgepole	8			
red	8			Use Southern Yellow Pine Specification 36.
southern yellow	8			36.
Posts, fence, round:				
Cedar	5	5	6	No standard specification.
Douglas fir	6	6	7	No post specification. Use Pole Specification 51.
Pine, all species	6	6	7	37.
Lumber and structural timber 5 in. or less in thickness (nominal dimensions) for land or fresh water use:				
Douglas fir	10	10	12	38.
Pine, southern yellow	10	10		53.
Structural timber more than 5 in. thick (nominal dimensions) for land or fresh water use:				
Douglas fir	8	8	10	38.
Pine, southern yellow	10	10		53.
Lumber and structural timber for use in salt water:				
Douglas fir	14	14		38.
Pine, southern yellow	20	20		53.

Table 62. Specifications for the Preservative Treatment of Wood—*Continued*

PRESERVATIVE SALTS

Form of product and service	Minimum net retention of preservative recommended					Treating specification (the current issue of A. W. P. A. Standard Specification listed)
	Zinc chloride (current issue of Fed. Spec. No. TT-W-576, and A. W. P. A. Spec. 17)	Celcure (acid cupric chromate) (current issue of Fed. Spec. TT-W-546) (No A. W. P. A. Spec.)	Chromated zinc chloride (current issue of Fed. Spec. TT-W-551) (No A. W. P. A. Spec.)	Wolman salt Tanalith (current issue of Fed. Spec. TT-W-573) (No A. W. P. A. Spec.)	Zinc meta arsenite (current issue of Fed. Spec. TT-W-581) (No A. W. P. A. Spec.)	
	Lb. dry salt per cu. ft.	*Lb. dry salt per cu. ft.*	*Lb. dry salt per cu. ft.*	*Lb. dry salt per cu. ft.*	*Lb. dry salt per cu. ft.*	
Lumber and structural timber:						
Dougas fir	1	0.50	0.75	0.35	0.35	38
Pine, southern yellow	1	.50	.75	.35	.35	53

APPENDIXES

APPENDIX A

Sizes Based on American Lumber Standards, and Properties of Boards, Dimension, and Timbers

Nominal Size b ↓ in Inches h ↓	American Standard Dressed Size b ↓ (s 4 s) in Inches h ↓	Area of Section $A = bh$ Sq. In.	Moment of Inertia $I = \frac{bh^3}{12}$	Section Modulus $S = \frac{bh^2}{6}$
1×3	$\frac{25}{32} \times 2\frac{5}{8}$	2.05	1.18	0.90
1×4	$\frac{25}{32} \times 3\frac{5}{8}$	2.83	3.10	1.71
1×5	$\frac{25}{32} \times 4\frac{5}{8}$	3.61	6.44	2.79
1×6	$\frac{25}{32} \times 5\frac{5}{8}$	4.39	11.59	4.12
1×8	$\frac{25}{32} \times 7\frac{1}{2}$	5.86	27.47	7.32
1×10	$\frac{25}{32} \times 9\frac{1}{2}$	7.42	55.82	11.75
1×12	$\frac{25}{32} \times 11\frac{1}{2}$	8.98	99.02	17.22
$1\frac{1}{4} \times 3$	$1\frac{1}{16} \times 2\frac{5}{8}$	2.79	1.60	1.22
$1\frac{1}{4} \times 4$	$1\frac{1}{16} \times 3\frac{5}{8}$	3.85	4.22	2.33
$1\frac{1}{4} \times 5$	$1\frac{1}{16} \times 4\frac{5}{8}$	4.91	8.76	3.79
$1\frac{1}{4} \times 6$	$1\frac{1}{16} \times 5\frac{5}{8}$	5.98	15.76	5.60
$1\frac{1}{4} \times 8$	$1\frac{1}{16} \times 7\frac{1}{2}$	7.97	37.35	9.96
$1\frac{1}{4} \times 10$	$1\frac{1}{16} \times 9\frac{1}{2}$	10.09	75.91	15.98
$1\frac{1}{4} \times 12$	$1\frac{1}{16} \times 11\frac{1}{2}$	12.22	134.66	23.42
$1\frac{1}{2} \times 3$	$1\frac{5}{16} \times 2\frac{5}{8}$	3.45	1.98	1.51
$1\frac{1}{2} \times 4$	$1\frac{5}{16} \times 3\frac{5}{8}$	4.76	5.21	2.87
$1\frac{1}{2} \times 5$	$1\frac{5}{16} \times 4\frac{5}{8}$	6.07	10.82	4.68
$1\frac{1}{2} \times 6$	$1\frac{5}{16} \times 5\frac{5}{8}$	7.38	19.47	6.92
$1\frac{1}{2} \times 8$	$1\frac{5}{16} \times 7\frac{1}{2}$	9.84	46.14	12.30
$1\frac{1}{2} \times 10$	$1\frac{5}{16} \times 9\frac{1}{2}$	12.47	93.78	19.74
$1\frac{1}{2} \times 12$	$1\frac{5}{16} \times 11\frac{1}{2}$	15.09	166.35	28.93
2×2	$1\frac{5}{8} \times 1\frac{5}{8}$	2.64	0.58	0.72
2×3	$1\frac{5}{8} \times 2\frac{5}{8}$	4.27	2.45	1.87
2×4	$1\frac{5}{8} \times 3\frac{5}{8}$	5.89	6.45	3.56
2×5	$1\frac{5}{8} \times 4\frac{5}{8}$	7.52	13.40	5.79

Sizes Based on American Lumber Standards, and Properties of Boards, Dimension, and Timbers—*Continued*

Nominal Size $b\downarrow$ in Inches $h\downarrow$	American Standard Dressed Size (s 4 s) $b\downarrow$ in Inches $h\downarrow$	Area of Section $A = bh$ Sq. In.	Moment of Inertia $I = \frac{bh^3}{12}$	Section Modulus $S = \frac{bh^2}{6}$
2×6	$1\frac{5}{8} \times 5\frac{5}{8}$	9.14	24.10	8.57
2×7	$1\frac{5}{8} \times 6\frac{5}{8}$	10.77	39.38	11.89
2×8	$1\frac{5}{8} \times 7\frac{1}{2}$	12.19	57.13	15.23
2×9	$1\frac{5}{8} \times 8\frac{1}{2}$	13.81	83.16	19.57
2×10	$1\frac{5}{8} \times 9\frac{1}{2}$	15.44	116.10	24.44
2×12	$1\frac{5}{8} \times 11\frac{1}{2}$	18.69	205.95	35.82
2×14	$1\frac{5}{8} \times 13\frac{1}{2}$	21.94	333.18	49.36
2×16	$1\frac{5}{8} \times 15\frac{1}{2}$	25.19	504.27	65.07
2×18	$1\frac{5}{8} \times 17\frac{1}{2}$	28.44	725.75	82.94
2×20	$1\frac{5}{8} \times 19\frac{1}{2}$	31.69	1,004.10	102.98
$2\frac{1}{2} \times 4$	$2\frac{1}{8} \times 3\frac{5}{8}$	7.70	8.44	4.65
$2\frac{1}{2} \times 5$	$2\frac{1}{8} \times 4\frac{5}{8}$	9.83	17.52	7.58
$2\frac{1}{2} \times 6$	$2\frac{1}{8} \times 5\frac{5}{8}$	11.95	31.52	11.21
$2\frac{1}{2} \times 7$	$2\frac{1}{8} \times 6\frac{5}{8}$	14.08	51.49	15.54
$2\frac{1}{2} \times 8$	$2\frac{1}{8} \times 7\frac{1}{2}$	15.94	74.71	19.92
$2\frac{1}{2} \times 9$	$2\frac{1}{8} \times 8\frac{1}{2}$	18.06	108.75	25.59
$2\frac{1}{2} \times 10$	$2\frac{1}{8} \times 9\frac{1}{2}$	20.19	151.83	31.96
$2\frac{1}{2} \times 12$	$2\frac{1}{8} \times 11\frac{1}{2}$	24.44	269.32	46.84
$2\frac{1}{2} \times 14$	$2\frac{1}{8} \times 13\frac{1}{2}$	28.69	435.69	64.55
$2\frac{1}{2} \times 16$	$2\frac{1}{8} \times 15\frac{1}{2}$	32.94	659.44	85.09
$2\frac{1}{2} \times 18$	$2\frac{1}{8} \times 17\frac{1}{2}$	37.19	949.06	108.46
$2\frac{1}{2} \times 20$	$2\frac{1}{8} \times 19\frac{1}{2}$	41.44	1,313.05	134.67
3×1	$2\frac{5}{8} \times \frac{25}{32}$	2.05	0.10	0.27
$3 \times 1\frac{1}{4}$	$2\frac{5}{8} \times 1\frac{1}{16}$	2.79	.26	.49
$3 \times 1\frac{1}{2}$	$2\frac{5}{8} \times 1\frac{5}{16}$	3.45	.49	.75
3×2	$2\frac{5}{8} \times 1\frac{5}{8}$	4.27	.94	1.16
$3 \times 2\frac{1}{2}$	$2\frac{5}{8} \times 2\frac{1}{8}$	5.58	2.10	1.98
3×3	$2\frac{5}{8} \times 2\frac{5}{8}$	6.89	3.96	3.01
3×4	$2\frac{5}{8} \times 3\frac{5}{8}$	9.52	10.42	5.75
3×5	$2\frac{5}{8} \times 4\frac{5}{8}$	12.14	21.64	9.36

Sizes Based on American Lumber Standards, and Properties of Boards, Dimension, and Timbers—*Continued*

Nominal Size b ↓ in Inches h ↓	American Standard Dressed Size b ↓ (s 4 s) in Inches h ↓	Area of Section $A = bh$ Sq. In.	Moment of Inertia $I = \frac{bh^3}{12}$	Section Modulus $S = \frac{bh^2}{6}$
3 × 6	$2\frac{5}{8} \times 5\frac{5}{8}$	14.77	38.93	13.84
3 × 7	$2\frac{5}{8} \times 6\frac{5}{8}$	17.39	63.61	19.20
3 × 8	$2\frac{5}{8} \times 7\frac{1}{2}$	19.69	92.29	24.61
3 × 9	$2\frac{5}{8} \times 8\frac{1}{2}$	22.31	134.34	31.61
3 × 10	$2\frac{5}{8} \times 9\frac{1}{2}$	24.94	187.55	39.48
3 × 12	$2\frac{5}{8} \times 11\frac{1}{2}$	30.19	332.69	57.86
3 × 14	$2\frac{5}{8} \times 13\frac{1}{2}$	35.44	538.21	79.73
3 × 16	$2\frac{5}{8} \times 15\frac{1}{2}$	40.69	814.60	105.11
3 × 18	$2\frac{5}{8} \times 17\frac{1}{2}$	45.94	1,172.36	133.98
3 × 20	$2\frac{5}{8} \times 19\frac{1}{2}$	51.19	1,622.00	166.36
4 × 1	$3\frac{5}{8} \times \frac{25}{32}$	2.83	0.14	0.37
4 × $1\frac{1}{4}$	$3\frac{5}{8} \times 1\frac{1}{16}$	3.85	.36	.68
4 × $1\frac{1}{2}$	$3\frac{5}{8} \times 1\frac{5}{16}$	4.76	.68	1.04
4 × 2	$3\frac{5}{8} \times 1\frac{5}{8}$	5.89	1.30	1.60
4 × $2\frac{1}{2}$	$3\frac{5}{8} \times 2\frac{1}{8}$	7.70	2.90	2.73
4 × 3	$3\frac{5}{8} \times 2\frac{5}{8}$	9.52	5.46	4.16
4 × 4	$3\frac{5}{8} \times 3\frac{5}{8}$	13.14	14.39	7.94
4 × 5	$3\frac{5}{8} \times 4\frac{5}{8}$	16.77	29.89	12.92
4 × 6	$3\frac{5}{8} \times 5\frac{5}{8}$	20.39	53.76	19.12
4 × 7	$3\frac{5}{8} \times 6\frac{5}{8}$	24.02	87.84	26.52
4 × 8	$3\frac{5}{8} \times 7\frac{1}{2}$	27.19	127.44	33.98
4 × 9	$3\frac{5}{8} \times 8\frac{1}{2}$	30.81	185.52	43.65
4 × 10	$3\frac{5}{8} \times 9\frac{1}{2}$	34.44	259.00	54.53
4 × 12	$3\frac{5}{8} \times 11\frac{1}{2}$	41.69	459.43	79.90
4 × 14	$3\frac{5}{8} \times 13\frac{1}{2}$	48.94	743.24	110.11
4 × 16	$3\frac{5}{8} \times 15\frac{1}{2}$	56.19	1,124.92	145.15
4 × 18	$3\frac{5}{8} \times 17\frac{1}{2}$	63.44	1,618.98	185.03
4 × 20	$3\frac{5}{8} \times 19\frac{1}{2}$	70.69	2,239.91	229.73
5 × 1	$4\frac{5}{8} \times \frac{25}{32}$	3.61	0.18	0.47
5 × $1\frac{1}{4}$	$4\frac{5}{8} \times 1\frac{1}{16}$	4.91	.46	.87
5 × $1\frac{1}{2}$	$4\frac{5}{8} \times 1\frac{5}{16}$	6.07	.87	1.33
5 × 2	$4\frac{5}{8} \times 1\frac{5}{8}$	7.52	1.65	2.04

Sizes Based on American Lumber Standards, and Properties of Boards, Dimension, and Timbers—*Continued*

Nominal Size $b \downarrow$ in Inches $h \downarrow$	American Standard Dressed Size $b \downarrow$ (s 4 s) in Inches $h \downarrow$	Area of Section $A = bh$ Sq. In.	Moment of Inertia $I = \frac{bh^3}{12}$	Section Modulus $S = \frac{bh^2}{6}$
$5 \times 2\frac{1}{2}$	$4\frac{5}{8} \times 2\frac{1}{8}$	9.83	3.70	3.48
5×3	$4\frac{5}{8} \times 2\frac{5}{8}$	12.14	6.97	5.31
5×4	$4\frac{5}{8} \times 3\frac{5}{8}$	16.77	18.36	10.13
5×5	$4\frac{1}{2} \times 4\frac{1}{2}$	20.25	34.17	15.19
5×6	$4\frac{1}{2} \times 5\frac{1}{2}$	24.75	62.39	22.69
5×7	$4\frac{1}{2} \times 6\frac{1}{2}$	29.25	102.98	31.69
5×8	$4\frac{1}{2} \times 7\frac{1}{2}$	33.75	158.20	42.19
5×9	$4\frac{1}{2} \times 8\frac{1}{2}$	38.25	230.30	54.19
5×10	$4\frac{1}{2} \times 9\frac{1}{2}$	42.75	321.52	67.69
5×12	$4\frac{1}{2} \times 11\frac{1}{2}$	51.75	570.33	99.19
5×14	$4\frac{1}{2} \times 13\frac{1}{2}$	60.75	922.64	136.69
5×16	$4\frac{1}{2} \times 15\frac{1}{2}$	69.75	1,396.45	180.19
5×18	$4\frac{1}{2} \times 17\frac{1}{2}$	78.75	2,009.77	229.69
5×20	$4\frac{1}{2} \times 19\frac{1}{2}$	87.75	2,780.58	285.19
6×1	$5\frac{5}{8} \times \frac{25}{32}$	4.40	0.22	0.57
$6 \times 1\frac{1}{4}$	$5\frac{5}{8} \times 1\frac{1}{16}$	5.98	.56	1.06
$6 \times 1\frac{1}{2}$	$5\frac{5}{8} \times 1\frac{5}{16}$	7.38	1.06	1.62
6×2	$5\frac{5}{8} \times 1\frac{5}{8}$	9.14	2.01	2.48
$6 \times 2\frac{1}{2}$	$5\frac{5}{8} \times 2\frac{1}{8}$	11.95	4.50	4.23
6×3	$5\frac{5}{8} \times 2\frac{5}{8}$	14.77	8.48	6.46
6×4	$5\frac{5}{8} \times 3\frac{5}{8}$	20.39	22.33	12.32
6×5	$5\frac{1}{2} \times 4\frac{1}{2}$	24.75	41.77	18.56
6×6	$5\frac{1}{2} \times 5\frac{1}{2}$	30.25	76.26	27.73
6×7	$5\frac{1}{2} \times 6\frac{1}{2}$	35.75	125.87	38.73
6×8	$5\frac{1}{2} \times 7\frac{1}{2}$	41.25	193.36	51.56
6×9	$5\frac{1}{2} \times 8\frac{1}{2}$	46.75	281.47	66.23
6×10	$5\frac{1}{2} \times 9\frac{1}{2}$	52.25	392.96	82.73
6×12	$5\frac{1}{2} \times 11\frac{1}{2}$	63.25	697.07	121.23
6×14	$5\frac{1}{2} \times 13\frac{1}{2}$	74.25	1,127.67	167.06
6×16	$5\frac{1}{2} \times 15\frac{1}{2}$	85.25	1,706.78	220.23
6×18	$5\frac{1}{2} \times 17\frac{1}{2}$	96.25	2,456.38	280.73
6×20	$5\frac{1}{2} \times 19\frac{1}{2}$	107.25	3,398.48	348.56
6×22	$5\frac{1}{2} \times 21\frac{1}{2}$	118.25	4,555.09	423.73
6×24	$5\frac{1}{2} \times 23\frac{1}{2}$	129.25	5,948.19	506.23

Sizes Based on American Lumber Standards, and Properties of Boards, Dimension, and Timbers—*Continued*

Nominal Size $b \downarrow$ in Inches $h \downarrow$	American Standard Dressed Size (s 4 s) $b \downarrow$ in Inches $h \downarrow$	Area of Section $A = bh$ Sq. In.	Moment of Inertia $I = \frac{bh^3}{12}$	Section Modulus $S = \frac{bh^2}{6}$
7 × 2	$6\frac{5}{8} \times 1\frac{5}{8}$	10.77	2.37	2.92
$7 \times 2\frac{1}{2}$	$6\frac{5}{8} \times 2\frac{1}{8}$	14.08	5.30	4.99
7 × 3	$6\frac{5}{8} \times 2\frac{5}{8}$	17.39	9.99	7.61
7 × 4	$6\frac{5}{8} \times 3\frac{5}{8}$	24.02	26.30	14.51
7 × 5	$6\frac{1}{2} \times 4\frac{1}{2}$	29.25	49.36	21.94
7 × 6	$6\frac{1}{2} \times 5\frac{1}{2}$	35.75	90.12	32.77
7 × 7	$6\frac{1}{2} \times 6\frac{1}{2}$	42.25	148.76	45.77
7 × 8	$6\frac{1}{2} \times 7\frac{1}{2}$	48.75	228.52	60.94
7 × 9	$6\frac{1}{2} \times 8\frac{1}{2}$	55.25	332.65	78.27
7 × 10	$6\frac{1}{2} \times 9\frac{1}{2}$	61.75	464.41	97.77
7 × 12	$6\frac{1}{2} \times 11\frac{1}{2}$	74.75	823.81	143.27
8 × 1	$7\frac{1}{2} \times \frac{25}{32}$	5.86	0.30	0.76
$8 \times 1\frac{1}{4}$	$7\frac{1}{2} \times 1\frac{1}{16}$	7.97	.75	1.41
$8 \times 1\frac{1}{2}$	$7\frac{1}{2} \times 1\frac{5}{16}$	9.84	1.41	2.15
8 × 2	$7\frac{1}{2} \times 1\frac{5}{8}$	12.19	2.68	3.30
$8 \times 2\frac{1}{2}$	$7\frac{1}{2} \times 2\frac{1}{8}$	15.94	6.00	5.64
8 × 3	$7\frac{1}{2} \times 2\frac{5}{8}$	19.69	11.30	8.61
8 × 4	$7\frac{1}{2} \times 3\frac{5}{8}$	27.19	29.77	16.43
8 × 5	$7\frac{1}{2} \times 4\frac{1}{2}$	33.75	56.95	25.31
8 × 6	$7\frac{1}{2} \times 5\frac{1}{2}$	41.25	103.98	37.81
8 × 7	$7\frac{1}{2} \times 6\frac{1}{2}$	48.75	171.64	52.81
8 × 8	$7\frac{1}{2} \times 7\frac{1}{2}$	56.25	263.67	70.31
8 × 9	$7\frac{1}{2} \times 8\frac{1}{2}$	63.75	383.83	90.31
8 × 10	$7\frac{1}{2} \times 9\frac{1}{2}$	71.25	535.86	112.81
8 × 12	$7\frac{1}{2} \times 11\frac{1}{2}$	86.25	950.55	165.31
8 × 14	$7\frac{1}{2} \times 13\frac{1}{2}$	101.25	1,537.73	227.81
8 × 16	$7\frac{1}{2} \times 15\frac{1}{2}$	116.25	2,327.42	300.31
8 × 18	$7\frac{1}{2} \times 17\frac{1}{2}$	131.25	3,349.61	382.81
8 × 20	$7\frac{1}{2} \times 19\frac{1}{2}$	146.25	4,634.30	475.31
8 × 22	$7\frac{1}{2} \times 21\frac{1}{2}$	161.25	6,211.48	577.81
8 × 24	$7\frac{1}{2} \times 23\frac{1}{2}$	176.25	8,111.17	690.31

Sizes Based on American Lumber Standards, and Properties of Boards, Dimension, and Timbers—*Continued*

Nominal Size b ↓ in Inches h ↓	American Standard Dressed Size b ↓ (s 4 s) in Inches h ↓	Area of Section $A = bh$ Sq. In.	Moment of Inertia $I = \frac{bh^3}{12}$	Section Modulus $S = \frac{bh^2}{6}$
9 × 2	$8\frac{1}{2} \times 1\frac{5}{8}$	13.81	3.04	3.74
$9 \times 2\frac{1}{2}$	$8\frac{1}{2} \times 2\frac{1}{8}$	18.06	6.80	6.40
9 × 3	$8\frac{1}{2} \times 2\frac{5}{8}$	22.31	12.81	9.76
9 × 4	$8\frac{1}{2} \times 3\frac{5}{8}$	30.81	33.74	18.62
9 × 5	$8\frac{1}{2} \times 4\frac{1}{2}$	38.25	64.55	28.69
9 × 6	$8\frac{1}{2} \times 5\frac{1}{2}$	46.75	117.85	42.85
9 × 7	$8\frac{1}{2} \times 6\frac{1}{2}$	55.25	194.53	59.85
9 × 8	$8\frac{1}{2} \times 7\frac{1}{2}$	63.75	298.83	79.69
10 × 1	$9\frac{1}{2} \times \frac{25}{32}$	7.42	0.38	0.97
$10 \times 1\frac{1}{4}$	$9\frac{1}{2} \times 1\frac{1}{16}$	10.09	.95	1.79
$10 \times 1\frac{1}{2}$	$9\frac{1}{2} \times 1\frac{5}{16}$	12.47	1.79	2.73
10 × 2	$9\frac{1}{2} \times 1\frac{5}{8}$	15.44	3.40	4.18
$10 \times 2\frac{1}{2}$	$9\frac{1}{2} \times 2\frac{1}{8}$	20.19	7.60	7.15
10 × 3	$9\frac{1}{2} \times 2\frac{5}{8}$	24.94	14.32	10.91
10 × 4	$9\frac{1}{2} \times 3\frac{5}{8}$	34.44	37.71	20.81
10 × 5	$9\frac{1}{2} \times 4\frac{1}{2}$	42.75	72.14	32.06
10 × 6	$9\frac{1}{2} \times 5\frac{1}{2}$	52.25	131.71	47.90
10 × 7	$9\frac{1}{2} \times 6\frac{1}{2}$	61.75	217.41	66.90
10 × 8	$9\frac{1}{2} \times 7\frac{1}{2}$	71.25	333.98	89.06
10 × 10	$9\frac{1}{2} \times 9\frac{1}{2}$	90.25	678.76	142.90
10 × 12	$9\frac{1}{2} \times 11\frac{1}{2}$	109.25	1,204.03	209.40
10 × 14	$9\frac{1}{2} \times 13\frac{1}{2}$	128.25	1,947.80	288.56
10 × 16	$9\frac{1}{2} \times 15\frac{1}{2}$	147.25	2,948.07	380.40
10 × 18	$9\frac{1}{2} \times 17\frac{1}{2}$	166.25	4,242.84	484.90
10 × 20	$9\frac{1}{2} \times 19\frac{1}{2}$	185.25	5,870.11	602.06
10 × 22	$9\frac{1}{2} \times 21\frac{1}{2}$	204.25	7,867.88	731.90
10 × 24	$9\frac{1}{2} \times 23\frac{1}{2}$	223.25	10,274.15	874.40
10 × 26	$9\frac{1}{2} \times 25\frac{1}{2}$	242.25	13,126.92	1,029.56
10 × 28	$9\frac{1}{2} \times 27\frac{1}{2}$	261.25	16,464.19	1,197.40
10 × 30	$9\frac{1}{2} \times 29\frac{1}{2}$	280.25	20,323.96	1,377.90
12 × 1	$11\frac{1}{2} \times \frac{25}{32}$	8.98	0.46	1.17
$12 \times 1\frac{1}{4}$	$11\frac{1}{2} \times 1\frac{1}{16}$	12.22	1.15	2.16
$12 \times 1\frac{1}{2}$	$11\frac{1}{2} \times 1\frac{5}{16}$	15.09	2.17	3.30
12 × 2	$11\frac{1}{2} \times 1\frac{5}{8}$	18.69	4.11	5.06

Sizes Based on American Lumber Standards, and Properties of Boards, Dimension, and Timbers—*Continued*

Nominal Size b ↓ in Inches h ↓	American Standard Dressed Size b ↓ (s 4 s) in Inches h ↓	Area of Section $A = bh$ Sq. In.	Moment of Inertia $I = \frac{bh^3}{12}$	Section Modulus $S = \frac{bh^2}{6}$
12 × 2½	11½ × 2⅛	24.44	9.20	8.65
12 × 3	11½ × 2⅝	30.19	17.33	13.21
12 × 4	11½ × 3⅝	41.69	45.65	25.19
12 × 5	11½ × 4½	51.75	87.33	38.81
12 × 6	11½ × 5½	63.25	159.44	57.98
12 × 7	11½ × 6½	74.75	263.18	80.98
12 × 8	11½ × 7½	86.25	404.30	107.81
12 × 10	11½ × 9½	109.25	821.65	172.98
12 × 12	11½ × 11½	132.25	1,457.51	253.48
12 × 14	11½ × 13½	155.25	2,357.86	349.31
12 × 16	11½ × 15½	178.25	3,568.71	460.48
12 × 18	11½ × 17½	201.25	5,136.07	586.98
12 × 20	11½ × 19½	224.25	7,105.92	728.81
12 × 22	11½ × 21½	247.25	9,524.28	885.98
12 × 24	11½ × 23½	270.25	12,437.13	1,058.48
12 × 26	11½ × 25½	293.25	15,890.48	1,246.31
12 × 28	11½ × 27½	316.25	19,930.34	1,449.48
12 × 30	11½ × 29½	339.25	24,602.69	1,667.98
14 × 2	13½ × 1⅝	21.94	4.83	5.94
14 × 2½	13½ × 2⅛	28.69	10.80	10.16
14 × 3	13½ × 2⅝	35.44	20.35	15.50
14 × 4	13½ × 3⅝	48.94	53.59	29.57
14 × 5	13½ × 4½	60.75	102.52	45.56
14 × 6	13½ × 5½	74.25	187.17	68.06
14 × 8	13½ × 7½	101.25	474.61	126.56
14 × 10	13½ × 9½	128.25	964.55	203.06
14 × 12	13½ × 11½	155.25	1,710.98	297.56
14 × 14	13½ × 13½	182.25	2,767.92	410.06
14 × 16	13½ × 15½	209.25	4,189.36	540.56
14 × 18	13½ × 17½	236.25	6,029.30	689.06
14 × 20	13½ × 19½	263.25	8,341.73	855.56

Sizes Based on American Lumber Standards, and Properties of Boards, Dimension, and Timbers—*Continued*

Nominal Size b ↓ in Inches h ↓	American Standard Dressed Size (s 4 s) b ↓ in Inches h ↓	Area of Section $A = bh$ Sq. In.	Moment of Inertia $I = \frac{bh^3}{12}$	Section Modulus $S = \frac{bh^2}{6}$
14 × 22	13½ × 21½	290.25	11,180.67	1,040.06
14 × 24	13½ × 23½	317.25	14,600.11	1,242.56
14 × 26	13½ × 25½	344.25	18,654.05	1,463.06
14 × 28	13½ × 27½	371.25	23,396.48	1,701.56
14 × 30	13½ × 29½	398.25	28,881.42	1,958.06
16 × 3	15½ × 2⅝	40.69	23.36	17.80
16 × 4	15½ × 3⅝	56.19	61.53	33.95
16 × 5	15½ × 4½	69.75	117.70	52.31
16 × 6	15½ × 5½	85.25	214.90	78.15
16 × 8	15½ × 7½	116.25	544.92	145.31
16 × 10	15½ × 9½	147.25	1,107.44	233.15
16 × 12	15½ × 11½	178.25	1,964.46	341.65
16 × 14	15½ × 13½	209.25	3,177.98	470.81
16 × 16	15½ × 15½	240.25	4,810.01	620.65
16 × 18	15½ × 17½	271.25	6,922.53	791.15
16 × 20	15½ × 19½	302.25	9,577.55	982.31
16 × 22	15½ × 21½	333.25	12,837.07	1,194.15
16 × 24	15½ × 23½	364.25	16,763.09	1,426.65
16 × 26	15½ × 25½	395.25	21,417.61	1,679.81
16 × 28	15½ × 27½	426.25	26,862.63	1,953.65
16 × 30	15½ × 29½	457.25	33,160.15	2,248.15
18 × 4	17½ × 3⅝	63.44	69.47	38.33
18 × 5	17½ × 4½	78.75	132.89	59.06
18 × 6	17½ × 5½	96.25	242.63	88.23
18 × 8	17½ × 7½	131.25	615.23	164.06
18 × 10	17½ × 9½	166.25	1,250.34	263.23
18 × 12	17½ × 11½	201.25	2,217.94	385.73
18 × 14	17½ × 13½	236.25	3,588.05	531.56
18 × 16	17½ × 15½	271.25	5,430.65	700.73
18 × 18	17½ × 17½	306.25	7,815.76	893.23
18 × 20	17½ × 19½	341.25	10,813.36	1,109.06

Sizes Based on American Lumber Standards, and Properties of Boards, Dimension, and Timbers—*Continued*

Nominal Size b ↓ in Inches h ↓	American Standard Dressed Size (s 4 s) b ↓ in Inches h ↓	Area of Section $A = bh$ Sq. In.	Moment of Inertia $I = \frac{bh^3}{12}$	Section Modulus $S = \frac{bh^2}{6}$
18 × 22	17½ × 21½	376.25	14,493.46	1,348.23
18 × 24	17½ × 23½	411.25	18,926.07	1,610.73
18 × 26	17½ × 25½	446.25	24,181.17	1,896.56
18 × 28	17½ × 27½	481.25	30,328.78	2,205.73
18 × 30	17½ × 29½	516.25	37,438.88	2,538.23
20 × 4	19½ × 3⅝	70.69	77.41	42.71
20 × 5	19½ × 4½	87.75	148.08	65.81
20 × 6	19½ × 5½	107.25	270.36	98.31
20 × 8	19½ × 7½	146.25	685.55	182.81
20 × 10	19½ × 9½	185.25	1,393.23	293.31
20 × 12	19½ × 11½	224.25	2,471.42	429.81
20 × 14	19½ × 13½	263.25	3,998.11	592.31
20 × 16	19½ × 15½	302.25	6,051.30	780.81
20 × 18	19½ × 17½	341.25	8,708.98	995.31
20 × 20	19½ × 19½	380.25	12,049.17	1,235.81
20 × 22	19½ × 21½	419.25	16,149.86	1,502.31
20 × 24	19½ × 23½	458.25	21,089.05	1,794.81
20 × 26	19½ × 25½	497.25	26,944.73	2,113.31
20 × 28	19½ × 27½	536.25	33,794.92	2,457.81
20 × 30	19½ × 29½	575.25	41,717.61	2,828.31
22 × 6	21½ × 5½	118.25	298.09	108.40
22 × 8	21½ × 7½	161.25	755.86	201.56
22 × 10	21½ × 9½	204.25	1,536.13	323.40
22 × 12	21½ × 11½	247.25	2,724.90	473.90
22 × 14	21½ × 13½	290.25	4,408.17	653.06
22 × 16	21½ × 15½	333.25	6,671.94	860.90
22 × 18	21½ × 17½	376.25	9,602.21	1,097.40
22 × 20	21½ × 19½	419.25	13,284.98	1,362.56
22 × 22	21½ × 21½	462.25	17,806.26	1,656.40
22 × 24	21½ × 23½	505.25	23,252.03	1,978.90
22 × 26	21½ × 25½	548.25	29,708.30	2,330.06
22 × 28	21½ × 27½	591.25	37,261.07	2,709.90
22 × 30	21½ × 29½	634.25	45,996.34	3,118.40

Sizes Based on American Lumber Standards, and Properties of Boards, Dimension, and Timbers—*Continued*

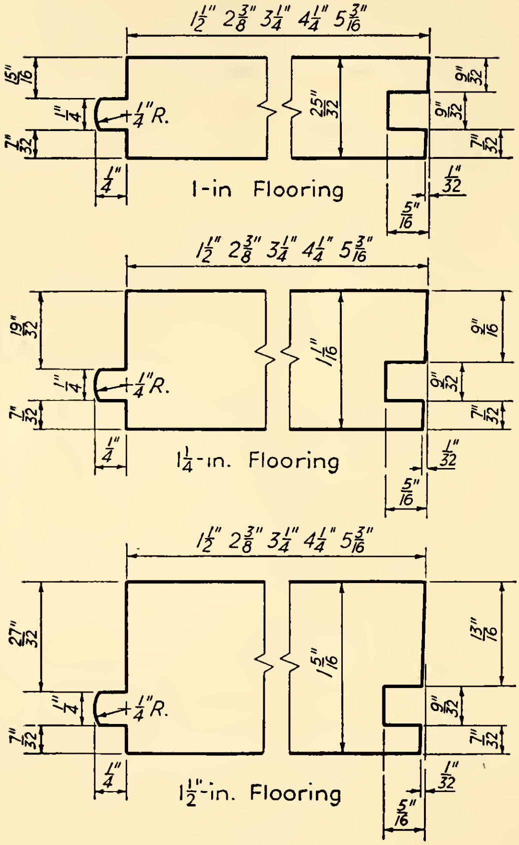
Nominal Size b ↓ in Inches h ↓	American Standard Dressed Size b ↓ (s 4 s) in Inches h ↓	Area of Section $A = bh$ Sq. In.	Moment of Inertia $I = \frac{bh^3}{12}$	Section Modulus $S = \frac{bh^2}{6}$
24 × 6	$23\frac{1}{2} \times 5\frac{1}{2}$	129.25	325.82	118.48
24 × 8	$23\frac{1}{2} \times 7\frac{1}{2}$	176.25	826.17	220.31
24 × 10	$23\frac{1}{2} \times 9\frac{1}{2}$	223.25	1,679.03	353.48
24 × 12	$23\frac{1}{2} \times 11\frac{1}{2}$	270.25	2,978.38	517.98
24 × 14	$23\frac{1}{2} \times 13\frac{1}{2}$	317.25	4,818.23	713.81
24 × 16	$23\frac{1}{2} \times 15\frac{1}{2}$	364.25	7,292.59	940.98
24 × 18	$23\frac{1}{2} \times 17\frac{1}{2}$	411.25	10,495.44	1,199.48
24 × 20	$23\frac{1}{2} \times 19\frac{1}{2}$	458.25	14,520.80	1,489.31
24 × 22	$23\frac{1}{2} \times 21\frac{1}{2}$	505.25	19,462.65	1,810.48
24 × 24	$23\frac{1}{2} \times 23\frac{1}{2}$	552.25	25,415.01	2,162.98
24 × 26	$23\frac{1}{2} \times 25\frac{1}{2}$	599.25	32,471.86	2,546.81
24 × 28	$23\frac{1}{2} \times 27\frac{1}{2}$	646.25	40,727.21	2,961.98
24 × 30	$23\frac{1}{2} \times 29\frac{1}{2}$	693.25	50,275.07	3,408.48
26 × 10	$25\frac{1}{2} \times 9\frac{1}{2}$	242.25	1,821.92	383.56
26 × 12	$25\frac{1}{2} \times 11\frac{1}{2}$	293.25	3,231.86	562.06
26 × 14	$25\frac{1}{2} \times 13\frac{1}{2}$	344.25	5,228.30	774.56
26 × 16	$25\frac{1}{2} \times 15\frac{1}{2}$	395.25	7,913.23	1,021.06
26 × 18	$25\frac{1}{2} \times 17\frac{1}{2}$	446.25	11,388.67	1,301.56
26 × 20	$25\frac{1}{2} \times 19\frac{1}{2}$	497.25	15,756.61	1,616.06
26 × 22	$25\frac{1}{2} \times 21\frac{1}{2}$	548.25	21,119.05	1,964.56
26 × 24	$25\frac{1}{2} \times 23\frac{1}{2}$	599.25	27,577.98	2,347.06
26 × 26	$25\frac{1}{2} \times 25\frac{1}{2}$	650.25	35,235.42	2,763.56
26 × 28	$25\frac{1}{2} \times 27\frac{1}{2}$	701.25	44,193.36	3,214.06
26 × 30	$25\frac{1}{2} \times 29\frac{1}{2}$	752.25	54,553.80	3,698.56
28 × 10	$27\frac{1}{2} \times 9\frac{1}{2}$	261.25	1,964.82	413.65
28 × 12	$27\frac{1}{2} \times 11\frac{1}{2}$	316.25	3,485.34	606.15
28 × 14	$27\frac{1}{2} \times 13\frac{1}{2}$	371.25	5,638.36	835.31
28 × 16	$27\frac{1}{2} \times 15\frac{1}{2}$	426.25	8,533.88	1,101.15
28 × 18	$27\frac{1}{2} \times 17\frac{1}{2}$	481.25	12,281.90	1,403.65
28 × 20	$27\frac{1}{2} \times 19\frac{1}{2}$	536.25	16,992.42	1,742.81
28 × 22	$27\frac{1}{2} \times 21\frac{1}{2}$	591.25	22,775.44	2,118.65

Sizes Based on American Lumber Standards, and Properties of Boards, Dimension, and Timbers—*Continued*

Nominal Size b ↓ in Inches h ↓	American Standard Dressed Size b ↓ (s 4 s) in Inches h ↓	Area of Section $A = bh$ Sq. In.	Moment of Inertia $I = \frac{bh^3}{12}$	Section Modulus $S = \frac{bh^2}{6}$
28 × 24	27½ × 23½	646.25	29,740.96	2,531.15
28 × 26	27½ × 25½	701.25	37,998.98	2,980.31
28 × 28	27½ × 27½	756.25	47,659.51	3,466.15
28 × 30	27½ × 29½	811.25	58,832.53	3,988.65
30 × 10	29½ × 9½	280.25	2,107.71	443.73
30 × 12	29½ × 11½	339.25	3,738.82	650.23
30 × 14	29½ × 13½	398.25	6,048.42	896.06
30 × 16	29½ × 15½	457.25	9,154.53	1,181.23
30 × 18	29½ × 17½	516.25	13,175.13	1,505.73
30 × 20	29½ × 19½	575.25	18,228.23	1,869.56
30 × 22	29½ × 21½	634.25	24,431.84	2,272.73
30 × 24	29½ × 23½	693.25	31,903.94	2,715.23
30 × 26	29½ × 25½	752.25	40,762.55	3,197.06
30 × 28	29½ × 27½	811.25	51,125.65	3,718.23
30 × 30	29½ × 29½	870.25	63,111.26	4,278.73

APPENDIX B

Patterns—American Lumber Standards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1/4"
1/4"R.
7/32"
1/4"
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7/32"
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1 1/2" 2 3/8" 3 1/4" 4 1/4" 5 3/16"
19/32"
1/4"
1/4"R.
7/32"
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1 1/16"
9/16"
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7/32"
1/32"
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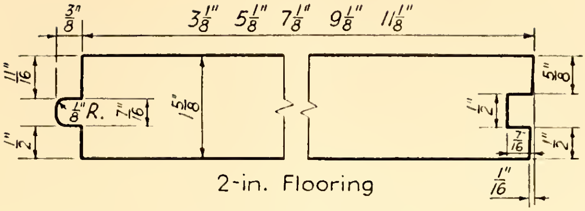
1 1/2" 2 3/8" 3 1/4" 4 1/4" 5 3/16"
27/32"
1/4"
1/4"R.
7/32"
1/4"
1 5/16"
13/16"
9/32"
7/32"
1/32"
5/16"

1-in Flooring

1 1/4-in. Flooring

1 1/2"-in. Flooring

Flooring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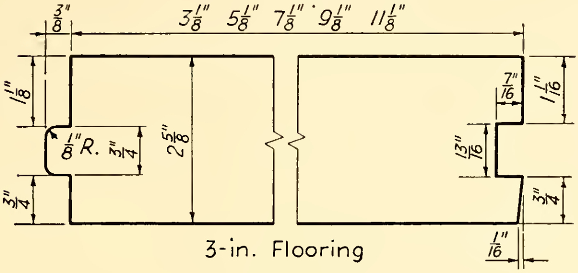
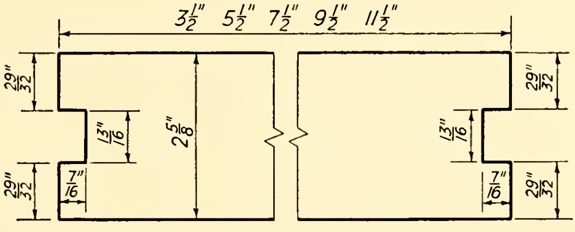
11/16"
1/8" R.
7/16"
1 5/8"
1/2"
5/8"
1/2"
7/16
1/2"
1/16"

2-in. Flooring


3/8"
3 1/8" 5 1/8" 7 1/8" 9 1/8" 11 1/8"
1 1/8"
1/8" R.
3/4"
2 5/8"
3/4"
7/16"
1 1/16"
13/16"
3/4"
1/16"

3-in. Flooring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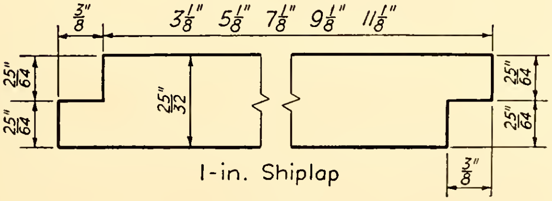
29/32"
13/16"
2 5/8"
7/16"
29/32"
29/32"
13/16"
7/16"
29/32"

3-in. Grooved for Splines

Heavy Flooring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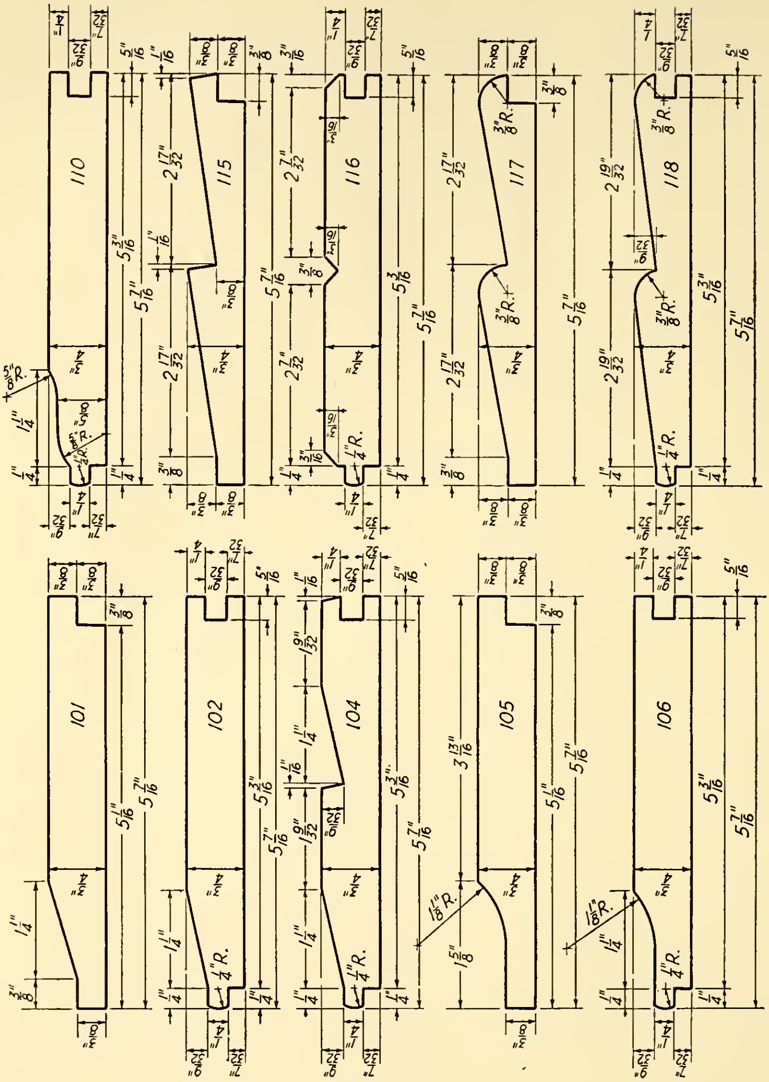
3 1/8" 5 1/8" 7 1/8" 9 1/8" 11 1/8"
25/64"
25/64"
25/32"
25/64"
25/64"
3/8"

1-in. Shiplap

Shiplap


110
115
116
117
118
101
102
104
105
106

3/4-in. by 6-in. drop siding

APPENDIX C

Definitions of Terms Used in Describing Standard Grades for Lumber

Air-dried. Dried by exposure to the atmosphere without artificial heat.

American Lumber Standards. American Lumber Standards embody basic provisions for softwood lumber dealing with recognized classifications, nomenclature, basic grades, seasoning standards, sizes, uniform workings, description, measurement, tally, shipping provisions, grade marking, tally cards, and inspection of lumber. The primary purpose of these standards is to serve as a guide or basic examples in the preparation or revision of the grading rules of the various lumber manufacturers' associations; their use as a framework for such rules will eliminate differences often existing.

Annual growth ring. *See* Ring, annual growth.

Bastard sawn. Hardwood lumber in which the annual rings make angles of 30° to 60° with the surface of the piece.

Beams and stringers. Lumber of rectangular cross section, 5 or more in. thick and 8 or more in. wide, graded with respect to its strength when loaded on the narrow face.

Bird's-eye. A small central spot with the wood fibers arranged around it in the form of an ellipse so as to give the appearance of an eye.

Boards. *See* Lumber.

Bow. A deviation flatwise from a straight line drawn from end to end of a piece. It is measured at the point of greatest distance from the straight line.

Boxed pith. When the pith is between the four faces on an end of a piece.

Brashness. A condition of wood characterized by low resistance to shock and by abrupt failure across the grain without normal splintering.

Broad-leaved trees. *See* Hardwoods.

Burl. A large wartlike excrescence on a tree trunk. It contains the dark piths of a large number of buds, which rarely develop.

Cambium. The layer of tissue just beneath the bark from which the new wood and bark cells of each year's growth develop.

Cell. A general term for the minute units of wood structure. It includes fibers, vessels, and other elements of diverse structure and functions.

Cellulose. The carbohydrate that is the principal constituent of wood and forms the framework of the cells.

Check. A lengthwise separation of the wood, which usually occurs across the rings of annual growth.

Close grain. *See* Grain.

Coarse grain. *See* Grain.

Conifer. *See* Softwood.

Connectors. *See* Timber Connectors.

Crook. A deviation edgewise from a straight line drawn from end to end of a piece. It is measured at the point of greatest distance from the straight line.

Cross break. A separation of the wood cells across the grain, such as may be due to tension resulting from unequal shrinkage or mechanical stress.

Cross grain. *See* Grain.

Cup. A curve in a piece across the grain or width of a piece. It is measured at the point of greatest deviation from a straight line drawn from edge to edge of a piece.

Decay. Disintegration of wood substance due to the action of wood-destroying fungi.

Incipient decay. The early stage of decay in which the disintegration has not proceeded far enough to soften or otherwise impair the hardness of the wood perceptibly.

Typical or advanced decay. The stage of decay in which the disintegration is readily recognized because the wood has become punky, soft, spongy, stringy, pitted, or crumbly.

Density. The mass of a body per unit volume. It is commonly although erroneously considered synonymous with specific gravity.

Density rule. Rules for estimating the density of wood based on percentage of summerwood and rate of growth.

Diagonal grain. *See* Grain.

Diffuse-porous woods. Hardwoods in which the pores are practically uniform in size throughout each annual ring, or decrease slightly in size toward the outer border of the ring.

Dimension. *See* Lumber.

Dimension stock. Squares or flat stock usually in pieces under the minimum sizes admitted in standard lumber grades, rough, dressed, green or dry, cut to the approximate dimension required for the various products of woodworking factories.

Dote. "Dote," "doze," and "rot" are synonymous with "decay."

Dry rot. A term loosely applied to many types of decay but especially to that which, when in an advanced stage, permits the wood to be easily crushed to a dry powder. The term is actually a misnomer for any decay, since all fungi require considerable moisture for growth, and the wood must have been moist at the time the "dry rot" occurred.

Durability. A general term for permanence or lastingness. Frequently used to refer to the degree of resistance of a species or of an individual piece of wood to decay. In this connection "resistance to decay" is a more specific term.

Edge grain. *See* Grain.

Empty-cell process. Any process for impregnating wood with preservatives or chemicals in which air is imprisoned in the wood under the pressure of the entering preservative and then expands, when the pressure is released, and drives out part of the injected preservative.

Encased knot. *See* Knot.

Extractives. Substances in wood, not an integral part of the cellular structure, that can be dissolved out with inert solvents.

Equilibrium moisture content. The moisture content at which wood neither gains nor loses moisture when surrounded by air at a given relative humidity and temperature.

Face width. The width of the face of a piece of dressed and matched or shiplapped lumber, not including the width of the tongue or lap. The amount of flooring, ceiling, siding, or other matched items required to cover a given area should be computed on the basis of the face width of the pieces. *See* Over-all width.

Factory and shop lumber. *See* Lumber.

Fiber. A wood fiber is a comparatively long (1/25 or less to 1/3 in.), narrow, tapering cell closed at both ends.

Fiber-saturation point. The stage in the drying or in the wetting of wood at which the cell walls are saturated and the cell cavities are free from water.

Figure. The pattern produced in a wood surface by irregular coloration and by annual growth rings, rays, knots, and such deviations from regular grain as interlocked and wavy grain.

Fireproofing. Making wood resistant to fire to a degree that makes it difficult to ignite and keeps it from supporting its own combustion.

Flat grain. *See* Grain, Plain sawn.

Flitch. A thick piece of lumber with wane (bark) on one or more edges.

Full-cell process. Any process for impregnating wood with preservatives or chemicals in which a vacuum is drawn to remove air from the wood before admitting the preservative.

Grade. Any of the quality classes into which products are segregated.

Grain. The direction, size, arrangement, appearance, or quality of the fibers in wood.

Close-grained wood. Wood with narrow and inconspicuous annual rings. The term is sometimes used to designate wood having small and closely spaced pores, but in this sense the term "fine-textured" is more often used.

Coarse-grained wood. Wood with wide and conspicuous annual rings in which there is considerable difference between springwood and summerwood. The term is also used to designate wood with large pores, but in this sense the term "coarse-textured" is more often used.

Cross grain. Wood in which the cells or fibers do not run parallel with the axis or sides of a piece.

Diagonal grain. Annual rings at an angle with the axis of a piece as a result of sawing at an angle with the bark of the tree.

Edge grain (vertical grain). Lumber in which the rings (so-called grain) form an angle of 45° or more with the surface of the piece.

Flat grain. Lumber in which the rings form an angle of less than 45° with the surface of the piece.

Heartwood. The inner core of the tree trunk comprising the annual rings containing non-living elements; usually darker in color than sapwood.

Interlocked grained wood. Wood in which the fibers are inclined in one direction in a number of rings of annual growth, then gradually reverse and are inclined in an opposite direction in succeeding growth rings, then reverse again.

Open-grained wood. Common classification of painters for wood with large pores, also known as "coarse-textured."

Plain sawn. Another term for flat grain, and used generally in hardwoods.

Quarter sawn. Another term for edge grain, and used generally in hardwoods.

Spiral grain. A type of growth in which the fibers take a spiral course about the bole of a tree instead of the normal vertical course. The spiral may extend right handed or left handed around the tree trunk.

Vertical grain. Another term for edge grain.

Wavy-grained wood. Wood in which the fibers collectively take the form of waves or undulations.

Green. Unseasoned, wet.

Growth ring. *See* Ring, annual growth.

Hardwoods. The botanical group of trees that are broadleaved. The term has no reference to the actual hardness of the wood.

Heart, heartwood. The wood, the cells of which no longer participate in the life processes of the tree, extending from the pith to the sapwood. Heartwood is usually darker in color than sapwood.

Imperfect manufacture. Includes all defects or blemishes which are produced in manufacturing, such as chipped grain, loosened grain, raised grain, torn grain, skips in dressing, hit or miss, variation in sawing, miscut lumber, machine burn, machine gouge, mismatching, and insufficient tongue or groove.

Interlocking grain. *See* Grain.

Joists and planks. Lumber of rectangular cross section, 2 in. to but not including 5 in. thick and 4 or more in. wide, graded with respect to its strength in bending when loaded either on the narrow face as joist or on the wide face as plank.

Kiln dried. *See* Seasoning.

Knot. A branch or limb, embedded in the tree, which has been cut through in the process of lumber manufacture. Knots are classified according to size, form, quality, and occurrence. To determine the size of a knot average maximum length and maximum width unless otherwise specified.

Encased knot. One whose rings of annual growth are not intergrown and homogeneous with those of the surrounding wood. The encasement may be partial or complete; of pitch or bark.

Intergrown knot. A knot whose rings of annual growth are completely intergrown with those of the surrounding wood.

Laminated wood. An assembly of wood built up of plies or laminations that have been joined either with glue or with mechanical fastenings. The term is most frequently applied where the plies are too thick to be classified as veneer and where the grain of all plies is parallel.

Lignin. A principal constituent of wood, second in quantity to cellulose. It incrusts the cell walls and cements the cells together.

Lumber. Lumber is the product of the saw and planing mill not further manufactured than by sawing, resawing, and passing lengthwise through a standard planing mill, cross-cutting to length, and working. Lumber of thickness not in excess of ¼ in. to be used for veneering is classified as veneer.

Factory and shop lumber. Lumber intended to be cut up for use in further manufacture. It is graded on the basis of the percentage of the area which will produce a limited number of cuttings of a specified, or a given minimum, size and quality.

Yard lumber. Lumber of all sizes and patterns which is intended for general building purposes. The grading of yard lumber is based on the intended use of the particular grade and is applied to each piece with reference to its size and length when graded without consideration to further manufacture.

STRIPS. Yard lumber less than 2 in. thick and less than 8 in. wide.

BOARDS. Yard lumber less than 2 in. thick and 8 or more in. wide.

DIMENSION. All yard lumber except boards, strips and timbers; that is yard lumber from 2 in. to but not including 5 in. thick, and of any width.

TIMBERS. Lumber 5 or more in. in least dimension.

Millwork. Generally all building materials made of finished wood and manufactured in millwork plants and planing mills are included under the term millwork, i.e., doors, window and door frames, sash, blinds, porch work, mantels, panel work, stairways, and special woodwork. It does not include finish dressed four sides, or siding, or partition, which are items of yard lumber.

Moisture content of wood. Weight of the water contained in the wood usually expressed in percentage of the weight of the oven-dry wood.

Moisture gradient. A condition of graduated moisture content between the inner and outer portions of a material, such as wood, due to the losing or absorbing of moisture.

Moisture-proofing. Making wood resistant to change in moisture content, especially to entrance of moisture.

Open-grain. *See* Grain.

Over-all width. The total of a wide piece of dressed and matched or shiplapped lumber including the width of the tongue or lap. The amount of such lumber required to cover a given area should not be computed on the basis of the over-all width since the tongue or lap is the means of joining the pieces and does not "cover" any surface. *See* face width.

Peck (found in cedar and cypress). Channeled or pitted areas or pockets of localized decay.

Pitch pocket. A well-defined opening between rings of annual growth, usually containing, or which has contained, more or less pitch, either solid or liquid. Bark also may be present in the pocket.

Pith. The small soft core occurring in the structural center of a log. The wood immediately surrounding the pith often contains small checks, shakes, or numerous pin knots, and is discolored; any such combination of characteristics is known as heart center.

Plywood. A piece of wood made of three or more layers of veneer joined with glue, and usually laid with the grain of adjoining plies at right angles. Almost always an odd number of plies are used to secure balanced construction.

Plain sawn. *See* Grain.

Planing mill products. Products worked to pattern, such as flooring, ceiling, and siding.

Pocket rot. Typical decay which appears in the form of a hole, pocket, or area of soft rot usually surrounded by apparently sound wood.

Pore. *See* Vessel.

Posts and timbers. Lumber of square or approximately square cross section, 5 by 5 in. and larger, graded primarily for use as posts or columns carrying longitudinal load but adapted for miscellaneous uses in which strength in bending is not especially important.

Preservative. With reference to wood, any substance applied to or injected into wood to protect it from attack of fungi, insects, or marine animals.

Quarter sawn. *See* Grain.

Radial. Coincident with a radius from the axis of the tree or log to the circumference.

Rate of growth. The rate at which a tree has laid on wood, measured radially in the trunk or in lumber cut from the trunk. The unit of measure in use is the number of annual growth rings per inch.

Rays, wood. Strips of cells extending radially within a tree and varying in height from a few cells in some species to 4 in. or more in oak. The rays serve primarily to store food and transport it horizontally in the tree.

Ring, annual growth. The growth layer put on in a single growth year.

Ring-porous woods. A group of hardwoods in which the pores are comparatively large at the beginning of each annual ring and decrease in size more or less abruptly toward the outer portion of the ring, thus forming a distinct inner zone of pores known as the springwood and the outer zone with smaller pores known as the summerwood.

Rot. *See* Decay.

Sap. All the fluids in a tree, except special secretions and excretions, such as oleoresin.

Sapwood. The outer layers of growth in a tree, exclusive of bark, which contains living elements; usually lighter in color than heartwood. Under most conditions sapwood is more susceptible to decay than heartwood; as a rule, it is more permeable to liquids than heartwood. Sapwood is not essentially weaker or stronger than heartwood of the same species.

Seasoning. The evaporation or extraction of moisture from green or partly dried wood.

Air dried or air seasoned. Dried by exposure to the atmosphere usually in a yard, without artificial heat.

Kiln-dried. Dried in a kiln with the use of artificial heat.

Second growth. Timber that has grown after the removal by any means of all or a large portion of the previous stand.

Shake. A separation along the grain, most of which occurs between the rings of annual growth.

Shop lumber. *See* Lumber.

Side-cut (pithless). The term used when the pith is not present in a piece.

Softwood. One of the group of trees which has needlelike or scalelike leaves, often referred to as conifers. The term "softwood" has no reference to the softness of the wood.

Specific gravity. The ratio of the weight of a body to the weight of an equal volume of water at some standard temperature.

Spiral grain. *See* Grain.

Split (through check). A lengthwise separation of the wood which occurs usually across the rings of annual growth, extending from one surface through the piece to the opposite surface, or to an adjoining surface.

Springwood. The more or less open and porous tissue marking the inner part of each annual ring, formed early in the period of growth. It is usually less dense and weaker mechanically than summerwood.

Stain, blue. A bluish or grayish discoloration of the sapwood caused by the growth of certain moldlike fungi on the surface and in the interior of the piece.

Strength. The properties of wood which enable it to resist different forces or loads. Strength may apply to any one of the mechanical properties, such as strength in bending, hardness, and strength in compression.

Strips. *See* Lumber.

Structural lumber. Lumber that is 2 or more in. thick and 4 or more in. wide, intended for use where working stresses are required. The grading of structural lumber is based on the strength of the piece and the use of the entire piece.

Summerwood. The dense, fibrous outer portion of each annual ring, usually without conspicuous pores, formed late in the growing period, not necessarily in summer. It is usually more dense and stronger mechanically than springwood.

Tangential. Strictly, coincident with a tangent at the circumference of a tree or log, or parallel to such a tangent. In practice, however, it often means roughly coincident with a growth ring.

Texture. A term often used interchangeably with grain.

Timber. A broad term including standing trees, and certain products thereof.

Round timber. Timber used in the original round form, such as poles, piling, and mine timbers.

Standing timber. Timber still on the stump.

Timber connectors. Rings and dowels of metal or wood in adjoining members, placed in pre-cut grooves or holes in timber framing, and the timber members then drawn and held tightly together by bolts. These devices provide an efficient means of transferring load at the joints of structural members.

Timbers. Lumber 5 in. or larger in least dimension.

Twist. A form of warp resulting in distortion caused by the turning or winding of the edges of a board.

Veneer. Thin sheets of wood.

Vertical grain. *See* Grain.

Vessels. Wood cells of comparatively large diameter which have open ends and are set one above the other, forming continuous tubes. The openings of the vessels on the surface of a piece of wood are usually referred to as pores.

Virgin growth. The original growth of mature trees.

Wane. Bark, or lack of wood or bark, from any cause, on edge or corner of a piece.

Warp. Any variation from a true or plane surface. It includes bow, crook, cup, or any combination thereof.

Wavy grain. *See* Grain.

Weathering. The mechanical and chemical disintegration and discoloration of the surface of wood that is caused by exposure to light, and by the alternate shrinking and swelling of the surface fibers with continual changes in moisture content due to weather changes. Weathering does not include decay.

Workability. The degree of ease and smoothness of cut obtainable with hand or machine tools.

Working of wood. Change in the dimensions of a piece of wood with change in moisture content.

Yard lumber. *See* Lumber.

APPENDIX D

Standard Lumber Abbreviations

AD. = Air dried.
a.l. = All lengths.
av. = Average.
av.w. = Average width.
av.l. = Average length.
a.w. = All widths.
B1S = Beaded one side.
B2S = Beaded two sides.
bd. = Board.
bd.ft. = Board foot; i.e., an area of 1 sq. ft. by 1 in. thick, or the product thereof.
bdl. = Bundle.
Bev. = Beveled.
b.m. = Board (foot) measure.
Btr. = Better.
Clg. = Ceiling.
Clr. = Clear.
CM. = Center matched; i.e., the tongue and groove joints are worked along the center of the edges of the piece.
Com. = Common.
Cag. = Casing.
cu.ft. = Cubic foot.
Cust. = Custom (sawed).
D&CM. *See* S&CM.
D&H = Dressed and headed; i.e., dressed one or two sides and worked to tongue and groove joints on both the edge and the ends.
D&M. = Dressed and matched; i.e., dressed one or two sides and tongued and grooved on the edges. The match may be center or standard.
D&SM. *See* S&SM.
D2S&CM. *See* S2S&CM.
D2S&M. *See* S2S&M.
D2S&SM. *See* S2S&SM.
Dim. = Dimension.
D.S. = Drop siding.
E. = Edge.
E&CB1S = Edge and center bead one side; surfaced one or two sides and with a longitudinal edge and center bead on a surfaced face.
E&CB2S = Edge and center bead two sides; all four sides surfaced and with a longitudinal edge and center bead on the two faces.
ECM = Ends center matched.
E&CV1S = Edge and center V one side; surfaced one or two sides and with a longitudinal edge and center V-shaped groove on a surfaced face.
E&CV2S = Edge and center V two sides; all four sides surfaced and with a longitudinal edge and center V-shaped groove on each of the two faces.
E.G. = Edge grain.
EM. = End matched—either center or standard.
ESM. = Ends standard matched.
FAS. = Firsts and seconds—a combined grade of the two upper grades of hardwoods.
f.bk. = Flat back.
fcty. = Factory (lumber).
F.G. = Flat grain.
Flg. = Flooring.

Frm. = Framing.
ft. = Foot or feet. Also one accent ('). *See* Symbols.
ft.b.m. = Feet board measure.
ft.s.m. = Feet surface measure.
G.R. = Grooved roofing.
H.bk. = Hollow back.
Hdwd. = Hardwood.
Hrt. = Heart.
Hrtwd. = Heartwood.
1s&2s. = Ones and twos—a combined grade of the hardwood grades of firsts and seconds.
in. = Inch or inches. Also two accent marks ("). *See* Symbols.
KD. = Kiln-dried.
k.d. = Knocked down.
lbr. = Lumber.
l.c.l. = Less carload lots.
lgth. = Length.
lgr. = Longer.
lin.ft. = Linear foot; i.e., 12 inches.
Lng. = Lining.
LR = Log run.
LR,MCO. = Log run, mill culls out.
Lth. = Lath.
M. = Thousand.
M.b.m. = Thousand (feet) board measure.
MCO. = Mill culls out.
Merch. = Merchantable.
m.l. = Mixed lengths.
Mldg. = Moulding.
MR. = Mill run.
M.s.m. = Thousand (feet) surface measure.
m.w. = Mixed widths.
No. = Number.
Ord. = Order.
P. = Planed.
Pat. = Pattern.
Pln. = Plain, as plain sawed.
Pn. = Partition.
Prod. = Production.
Qtd. = Quartered—when referring to hardwoods.
rdm. = Random.
res. = Resawed.
Rfg. = Roofing.
Rfrs. = Roofers.
rip. = Ripped.
r.l. = Random lengths.
rnd. = Round.
R.Sdg. = Rustic siding.
r.w. = Random widths.
S1E. = Surfaced one edge.
S2E. = Surfaced two edges.
S1S. = Surfaced one side.
S2S. = Surfaced two sides.
S1S1E. = Surfaced one side and one edge.
S2S1E. = Surfaced two sides and one edge.
S1S2E. = Surfaced one side and two edges.
S4S. = Surfaced four sides.
S4SCS. = Surfaced four sides with a calking seam on each edge.
S&CM. = Surfaced one or two sides and center matched.
S&M. = Surfaced and matched; i.e., surfaced one or two sides and tongued and grooved on the edges. The match may be center or standard.
S&SM. = Surfaced one or two sides and standard matched.
S2S&CM. = Surfaced two sides and center matched.
S2S&M. = Surfaced two sides and (center or standard) matched.
S2S&SM. = Surfaced two sides and standard matched.
Sap. = Sapwood.
SB. = Standard bead.
Sd. = Seasoned.
Sdg. = Siding.
Sel. = Select.
s.f. = Surface foot; i.e., an area of one square foot.
Sftwd. = Softwood.
Sh.D. = Shipping dry.
Ship. = Shipment or shipments.
Shlp. = Shiplap.
s.m. = Surface measure.

SM. = Standard matched.
snd. = Sound.
sq. = Square.
Sq.E. = Square edge.
sqrs. = Squares.
Std. = Standard.
stnd. = Stained.
stk. = Stock.
stp. = Stepping.
Symbols: ″ = inch or inches, as 12″.
′ = foot or feet, as 12′.
x = by, as a 6x8 timber.
When referring to the thickness of lumber, 4/4, 5/4, 6/4, 8/4, etc. = 1 inch, 1¼ inches, 1½ inches, 2 inches, etc.
T&G = Tongued and grooved.
TB&S. = Top, bottom, and sides.
Tbrs. = Timbers.
V1S. = V one side, i.e., a longitudinal V-shaped groove on one face of a piece of lumber.
V2S. = V two sides; i.e., a longitudinal V-shaped groove on two faces of a piece of lumber.
V.G. = Vertical grain.
w.a.l. = Wider, all lengths.
Wth. = Width.
wdr. = Wider.
wt. = Weight.

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Date Due

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FEB 12 '74			
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MAR 8 '80	MAR 10 '80		
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